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VOL. 8 MARCH, 1921 NO. 2


## TESTING WATER METERS IN SERIES[1]


BY J. E. GARRATT[2]


Will water meters show the same percentage of registration on test if tested singly or in series of two, three, five, ten or any number? Some will immediately say "No;" others will, perhaps, say, "Yes;" and some who say "Yes" might think they were wrong if it was left to a test to furnish the proof.

Meters are tested at different rates of flow, these different rates of flow being usually determined by orifices of different sizes. It is common practice for the meter testing man to refer to a test as "with a $\frac{1}{32}$-inch orifice" or "with a $\frac{1}{8}$-inch orifice" or any other size. He knows that a $\frac{1}{16}$-inch orifice allows more water to pass on the test in a given time than a $\frac{1}{32}$-inch orifice does and that a $\frac{1}{2}$-inch orifice passes more water than a $\frac{1}{4}$-inch, etc., but he considers that a certain size of orifice always gives the same amount of water through it in a given time. That is because he deals with a pressure in his particular city which is more or less steady and practically the same from day to day, but he forgets that that pressure of his may be 100 pounds per square inch while another city's may be only 40 pounds

[1] Presented at the Montreal Convention, June 23, 1920. Discussions are invited and should be sent to the Editor.

[2] Office Engineer, Board of Water Commissioners, Hartford, Conn.

per square inch and the flows through orifices, of course, are different with different pressures, in fact they vary about as the square root of the pressure, so that if the pressure is doubled the rate of water passing through the same size of orifice is increased about one-half. Consequently, a $\frac{1}{16}$-inch orifice in one meter testing room may mean two times or three times or one-half or any number of times or any part of what it does in some other testing room, provided the amounts of water passed by the orifice are expressed as a rate of so many gallons or cubic feet or pounds of water per minute or any other unit of time.

The first point, then, very evidently is that in all meter testing the various points at which the meters are tested should be designated as so many gallons or cubic feet or pounds per minute and not as at such and such an orifice.

If this is done and the same rates are produced when the meters are tested in series as when tested singly, the answer to the question which started this paper is "Yes, water meters will show the same percentage of registration on test whether tested singly or in series of any number."

The whole cause of the question in regard to this matter is due to this fact that the rates of flow, as already stated, are usually determined by orifices and the same orifices are used on the series test as on the single test, but the same orifices may not give the same rates of flow even in the same testing shop. Why? Because there may not be the same resulting pressure at the orifice.

When water passes through a meter at a certain rate there is a certain pressure loss in the meter which is different with different sizes and makes of meters. Also this pressure loss is different for different rates passing the meter and varies about as the square of the rate, that is, if the rate is doubled the pressure loss is about four times as great.

Now suppose a meter is tested by itself, using a $\frac{1}{8}$-inch orifice and a pressure of 70 pounds per square inch available at the inlet side of the meter. There are certain losses through the meter and piping, leaving a resulting pressure at the orifice of less than 70 pounds per square inch by the amount of the losses. The rate at which the meter is tested, is then, the flow through a $\frac{1}{8}$-inch orifice at this pressure of 70 pounds less the losses.

Now suppose four meters are tested in series. The total loss through the four meters is about four times what it was through the

one. Consequently the pressure at the orifice is less than on the single test by about three times the loss through one meter. Consequently the pressure at the orifice is less than before and since the amount of water passed by the orifice depends upon the pressure,

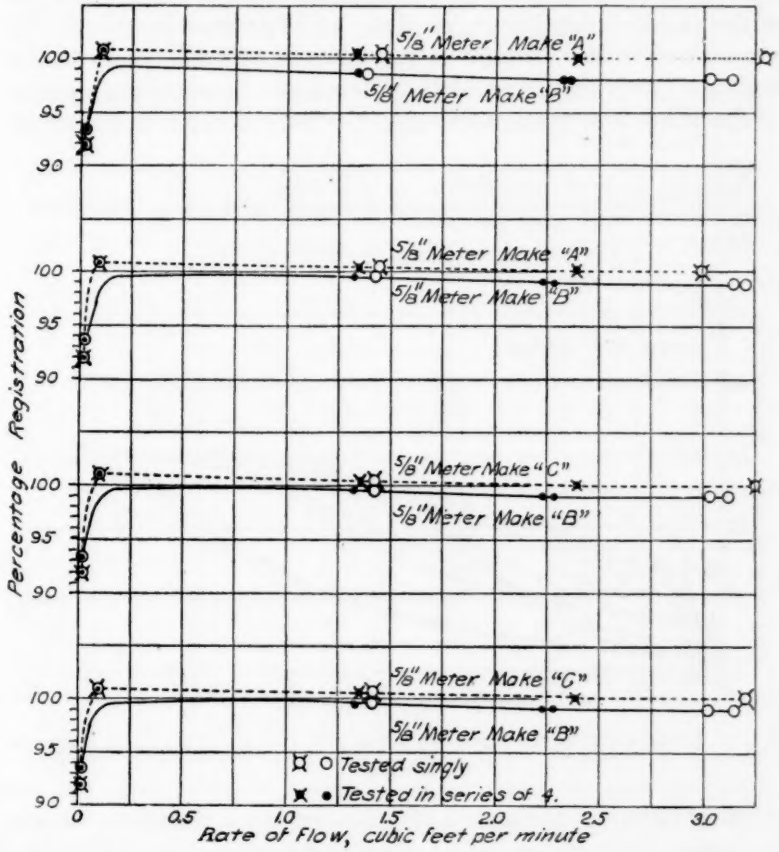



FIG. 1. CURVES SHOWING THAT RESULTS OF ACCURACY TESTS OF METERS ARE THE SAME WHEN TESTED SINGLY OR IN SERIES

the rate of flow at which the meter is tested will be less although the same orifice is used in each test. Such being the case, the two tests with the same orifice may not show the same percentage of registration because they are at different rates of flow.

Actually, it works out that way in water works meter testing only in exceptional cases, where many meters are tested together or where

extreme refinements of test are carried through. This is due to the form of the meter registration curve of test, as shown in figure 1. This curve rises quickly at a very low rate of flow to a few per cent below or above 100 and holds there with slight variation throughout the rest of its run. Tests with very small orifices which would lie in the rapidly rising low range of the curve produce practically the same rates of flow because at low rates of flow the losses in the meters are very small so that the difference in the resulting pressure at the orifice is very small and therefore the rates of flow are, for all

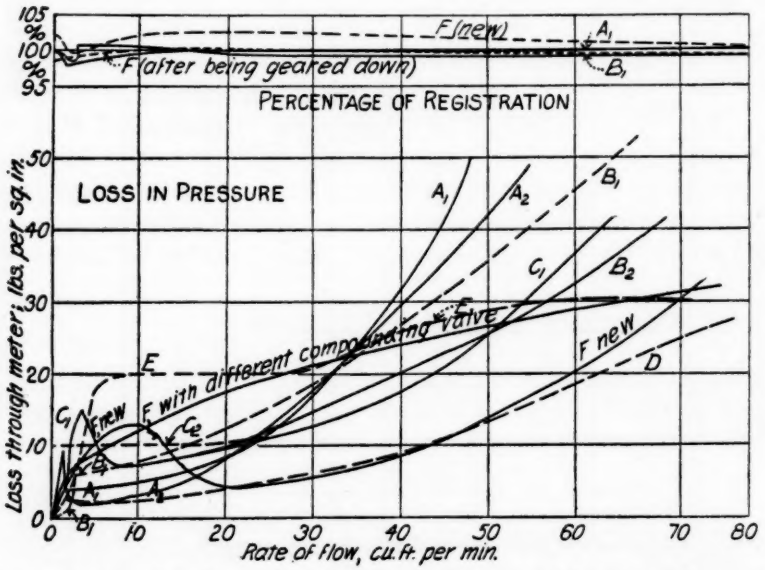



FIG. 2. LOSS IN PRESSURE THROUGH 3-INCH COMPOUND METERS OF VARIOUS MAKES

practical purposes, the same and the percentages of registration are the same. With larger orifices producing greater rates of flow, the losses in the meters are larger and become appreciable so that the resulting pressures at the orifices are different by the two methods of test and consequently the rates of flow through the meters are different. But the curve of registration usually varies so little and so gradually from the 100 per cent line, or a line parallel to the 100 per cent line, that the different rates of flow on the two tests are not so far different as to show different percentages of registration. This is very plainly shown in figure 1 and in table 1.

Of course the greater the number of meters tested in series as compared with the single test the greater the losses through the meters, the greater the difference in the resulting pressures at the orifice and the greater the difference in the rates of flow. The greater this difference in rate of flow, the greater the likelihood of different percentages of registration.

TABLE 1

*Percentage registration of meters tested singly and in series of four meters*

| METER | ORIFICE | TESTED SINGLY | | TESTED IN SERIES | |
|---|---|---|---|---|---|
| | | Rate | Registration | Rate | Registration |
| | *inch* | *cu. ft./min.* | *per cent* | *cu. ft./min.* | *per cent* |
| A | $\frac{1}{2}$ | 3.03 | 99.1 | 2.24 | 99.0 |
| | $\frac{1}{4}$ | 1.42 | 99.7 | 1.32 | 99.7 |
| | $\frac{1}{32}$ | 0.024 | 93.3 | 0.021 | 93.3 |
| A | $\frac{1}{2}$ | 3.03 | 99.1 | 2.24 | 99.0 |
| | $\frac{1}{4}$ | 1.43 | 99.7 | 1.32 | 99.7 |
| | $\frac{1}{32}$ | 0.024 | 93.3 | 0.021 | 93.3 |
| B | $\frac{1}{2}$ | 3.30 | 100.0 | 2.40 | 100.0 |
| | $\frac{1}{4}$ | 1.45 | 100.5 | 1.36 | 100.5 |
| | $\frac{1}{16}$ | 0.10 | 100.8 | 0.10 | 100.8 |
| | $\frac{1}{32}$ | 0.024 | 91.9 | 0.021 | 91.9 |
| C | $\frac{1}{2}$ | 3.28 | 100.0 | 2.40 | 100.0 |
| | $\frac{1}{4}$ | 1.42 | 100.5 | 1.36 | 100.5 |
| | $\frac{1}{16}$ | 0.10 | 100.8 | 0.10 | 100.8 |
| | $\frac{1}{32}$ | 0.022 | 91.9 | 0.022 | 91.9 |

The one conclusion to be drawn from this discussion is that all tests of meters should be referred to rates of flow through the meter in cubic feet or gallons or pounds per minute or hour and not to size of orifice. If this is done it makes no difference whether meters are tested singly or in tandem or in series of ten or of one hundred, the percentage of registration will always be the same at the same rate provided the mechanical condition of the meter has not changed.

# WATER BORNE TYPHOID FEVER AT GREENEVILLE, TENNESSEE[1]

By C. N. Harrub[2]

The water supply of Greeneville, Tennessee, is obtained from a spring which emerges from beneath a limestone ledge almost in the center of the city. It is located in the bottom of a natural basin toward which a large portion of the town drains. A concrete pool has been constructed at the spring. A portion of this pool is covered by the intake house, but about half of it is uncovered. From this place the water is conducted by a pipe line to another open pool at the pumping station. This second pool serves as a suction well, and the water is pumped from it to a standpipe, whence it flows by gravity to the city.

The sanitary survey shows a very unsatisfactory condition. As previously mentioned, the spring is in practically the lowest spot in town and is surrounded by dwellings on all sides. The concrete curb around the spring does not prevent the entrance of surface drainage which runs toward the spring from a large portion of the town. A concrete storm sewer has been constructed to discharge into the stream a short distance below the spring. A fence across the stream just below the end of the sewer so obstructs the flow in times of heavy rain that the storm water is backed into the spring.

At the pumping station the water supply is separated from a very filthy creek only by a concrete wall, through which two overflow pipes are placed. At times of high water in the creek these overflow pipes are sometimes submerged, and the only means employed to keep the creek from flowing directly into the pump suction well is to stuff cotton waste into the overflow pipes.

Since the establishment of the public water supply many private wells previously used have been abandoned and until such use was

[1] Discussion is invited and should be sent to the Editor.

[2] Associate Sanitary Engineer, U. S. Public Health Service; Acting State Sanitary Engineer, Nashville, Tenn.

finally forbidden by the city they were quite commonly converted into cesspools. One well so used was shown by test to be directly connected with the spring.

Until 1913 the water was used without any treatment. During that year the city experienced a severe epidemic of typhoid fever, which the evidence indicated was due to impure water. Analyses of the spring water indicated sewage contamination. A calcium hypochlorite plant was then installed to make the water safe, and undoubtedly was a very considerable factor in checking that epidemic. The original plant consisted of a grinder, mixing tank, two solution tanks and orifice box. The solution was fed through about 30 feet of half-inch pipe into the suction pool near the end of the suction main.

Early in May, 1920, typhoid fever began to appear with unusual frequency. However, until June 8 the only effort made to prevent the spread of the disease was the application of antityphoid vaccine to other members of the family where a case of typhoid occurred and exercising the usual care in disposing of excretions from the patient. On that date an appeal was made to the State Board of Health for assistance. The author, to whom the request was referred, immediately went to Greeneville, and in 24 hours was on the ground to investigate the epidemic and render any assistance possible in checking it.

The first step in the investigation was an examination of the water supply. This revealed the conditions above described, and also the fact that the hypochlorite plant was badly out of order. One of the solution tanks was completely out of commission, the bottom having fallen out. This made it necessary to use one solution tank continuously, feeding from it while making up the new batch of solution, and allowed no period for settling. The result of this was a variable strength solution, clogged feed lines and interrupted application of the solution to the water. The trouble was increased by occasional stirring of the solution.

It was also found that much less available chlorine was actually applied to the water than was thought. No records of the amount of hypochlorite used or frequency of preparing solutions were kept, and no definite information as to the dosage could be obtained. It was only by checking up the quantity purchased for the year that the average dose could be determined. This average dose, 0.07 p.p.m. available chlorine, was far too small to produce a safe water.

On account of the previous history of the water supply, which for the past eight years has always been contaminated, it was thought to be the probable cause of the present epidemic of typhoid fever. Consequently, one of the emergency chlorinators, recently furnished the State Board of Health by the Wallace & Tiernan Company, Inc., of New York, had been taken to Greeneville, and in order to accomplish efficient disinfection of the water it was considered necessary to install it. As the State Board of Health had up to that time been unable to obtain delivery of any liquid chlorine, an attempt had been made to obtain some in the vicinity of Greeneville. Through the courtesy of the City Health Officer of Kingsport a 100-pound cylinder of liquid chlorine was obtained from that place and put into service on June 12. The apparatus was set to feed the chlorine at a calculated rate of 0.6 p.p.m. on the basis of a half million gallons per day pumpage. Actual check showed the rate to be 0.58 p.p.m. Subsequent test indicated an excess of about 0.2 p.p.m. and the dose was consequently reduced to about 0.4 p.p.m. Bacteriological analyses conducted over a period of nine days indicated that the reduced amount was producing a practically sterile water with no fermentation in 10 cc. quantities of water planted in lactose broth, and it was continued without further change.

Up to the time of installing the chlorinator there had been 57 cases of typhoid fever and six deaths in a period of approximately nine weeks. During the eight days following 3 new cases were reported, the last one being reported on June 20. As the incubation period for typhoid fever is from twelve to fourteen days, it is apparent that no new infections could have occurred after the chlorinator was put in operation.

The dates of onset of the various cases are shown in table 1.

In addition to the 60 cases and six deaths recorded above, there was one case in the county outside of Greeneville which was fatal. This made a total of 61 cases and seven deaths during the epidemic. This corresponds to a morbidity rate of 2033 per 100,000 population, and a death rate of 233 per 100,000 population, and a mortality percentage of 11.5.

A complete investigation of the epidemic was made to determine any other possible source of the infection, but all the evidence collected pointed to the water supply as the responsible agent. The immediate checking of the epidemic by the proper treatment of the water supply fully confirmed this conclusion.

TABLE 1

*Cases of typhoid fever, Greeneville, Tennessee, 1920*

| DATE | CASES | WEEK ENDING | CASES FOR THE WEEK |
|---|---|---|---|
| April 6 | 1 | April 10 | 1 |
| April 15 | 2 | April 17 | 2 |
| April 20 | 1 | April 24 | 1 |
| April 26 | 1 | | |
| April 30 | 2 | | |
| May 1 | 5* | May 1 | 8 |
| May 2 | 2* | | |
| May 3 | 1 | | |
| May 4 | 2 | | |
| May 5 | 1* | | |
| May 7 | 3* | | |
| May 8 | 1 | May 8 | 10 |
| May 10 | 1 | | |
| May 14 | 1 | | |
| May 15 | 2* | May 15 | 4 |
| May 16 | 1 | | |
| May 17 | 1 | | |
| May 19 | 1 | | |
| May 21 | 1 | | |
| May 22 | 1 | May 22 | 5 |
| May 23 | 1 | | |
| May 27 | 1* | | |
| May 28 | 1 | | |
| May 29 | 1 | May 29 | 4 |
| May 30 | 5 | | |
| June 1 | 1 | | |
| June 2 | 1 | | |
| June 3 | 1 | | |
| June 4 | 2 | | |
| June 5 | 1 | June 5 | 11 |
| June 6 | 2 | | |
| June 7 | 4 | | |
| June 8 | 2 | | |
| June 10 | 2 | | |
| June 12 | 1 | June 12 | 11† |
| June 17 | 1 | | |
| June 18 | 1 | June 19 | 2 |
| June 20 | 1 | June 26 | 1 |
| Totals | 60 | | 60 |

* One of these cases died. Total, 6 deaths.

† Chlorinator installed June 12.

The typhoid fever history of Greene County, in which Greeneville is located, is given by districts for the years 1914–1920, inclusive, in table 2.

It is not possible to secure an accurate estimate of the population of the 10th District, which is the district in which Greeneville is located, of Greene County. However, the 1910 census gives Greene-

TABLE 2

*Deaths in Greene County from typhoid fever by districts*

| DISTRICTS | 1914 | 1915 | 1916 | 1917 | 1918 | 1919 | JANUARY–JULY, 1920 | TOTAL |
|---|---|---|---|---|---|---|---|---|
| 1st | 1 | – | 1 | – | – | – | – | 2 |
| 2nd | 1 | 1 | – | – | – | – | – | 2 |
| 3rd | – | 1 | – | 1 | – | – | – | 2 |
| 6th | – | 2 | – | – | – | – | – | 2 |
| 7th | 1 | 1 | – | – | 1 | 2 | 1 | 6 |
| 9th | 1 | – | – | – | – | – | – | 1 |
| 10th* | 1 | 2 | 1 | 3 | 3 | 3 | 7 | 20 |
| 11th | 1 | 3 | – | – | 3 | – | – | 7 |
| 12th | – | – | 1 | 2 | 2 | 1 | – | 6 |
| 14th | 1 | – | – | 1 | – | – | – | 2 |
| 15th | 1 | 2 | – | – | – | – | – | 3 |
| 16th | 1 | – | 2 | – | 1 | 2 | 1 | 7 |
| 17th | 1 | 1 | 1 | – | 1 | – | 1 | 5 |
| 18th | – | – | 2 | – | 1 | – | 3 | 6 |
| 19th | – | – | – | 1 | – | – | – | 1 |
| 20th | – | – | – | – | – | 1 | – | 1 |
| 21st | 1 | – | 1 | 4 | – | 1 | – | 7 |
| 22nd | – | – | 1 | – | 1 | – | 1 | 3 |
| 24th | – | – | – | 1 | – | – | – | 1 |
| 26th | 1 | – | – | – | – | – | – | 1 |
| Totals..... | 12 | 13 | 10 | 13 | 13 | 10 | 14 | 85 |

No deaths from typhoid reported in the 4th, 5th, 8th, 13th, 23rd and 25th districts.

* Includes Greeneville.

ville a population of 1920, and Greene County a population of 31,083. Hence, it is fair to assume that the 10th District does not contain more than 10 per cent of the county's total population.

In the period from January 1, 1914, to July 31, 1919, the records of the Bureau of Vital Statistics show that with the above estimated 10 per cent of the population the 10th District has had 23.5 per cent

of the total typhoid deaths in the county, or more than twice the county incidence.

No record of the epidemic of 1913 is available, as the Bureau of Vital Statistics was not established until 1914, and no local records were kept, but it was unofficially stated by officials and residents of Greeneville that there were about 120 cases at that time.

The study of this epidemic of water-borne typhoid fever should be of especial interest to water-works superintendents, as it indicates so pointedly the necessity of constant, careful watch over the operation of the disinfectant plant if efficient protection is to be afforded against a polluted water.

In justice to the city it must be added that as a result of the investigation by the State Board of Health, the city purchased a liquid chlorine apparatus and has taken steps to provide for filtration of the water supply.

# VALUATIONS OF THE IOWA CITY WATER COMPANY BASED ON PRESENT AND PRE-WAR COSTS[1]

By John H. Dunlap[2]

The Iowa City Water Company is a privately owned company. The population of Iowa City is about 12,000, exclusive of 4000 students in attendance at the University of Iowa. The plant was originally installed in 1882. The principal source of supply is from infiltration galleries estimated at about a half-mile in length beneath the bed of the Iowa River. Ten 8-inch flowing wells 160 feet deep contribute a small amount of the water used. At times when these two sources prove inadequate, some water is taken directly from the Iowa River, which is only about 100 feet from the plant.

*Plant statistics.*—The pumping equipment consists of a Holly quadruplex compound condensing crank and fly-wheel steam pump with a capacity of 1,500,000 gallons, installed in 1882; a Snow duplex direct-acting compound condensing steam pump with a capacity of 2,000,000 gallons, installed in 1889; and an Allis-Chalmers horizontal cross-compound crank and fly-wheel Corliss type high-duty steam pump with a capacity of 3,000,000 gallons, installed in 1914.

In 1909 an iron-removal plant was installed by the New York Continental Jewell Filtration Company. There are two sedimentation basins and four rectangular concrete gravity filter units with a capacity of 2,000,000 gallons per day. This plant, which was originally designed as an iron-removal plant, is now serving also as a filtration plant with coagulation with alum and lime supplemented by sterilization by liquid chlorine.

The distribution system consists of 26.4 miles of mains, 2 to 20 inches in diameter.

[1] Read before the Iowa Section, November 5, 1920. Discussions are invited and should be sent to the Editor.

[2] Professor of Hydraulics and Sanitary Engineering, State University of Iowa, Iowa City, Iowa.

*The valuations.*—Since 1915 four valuations of the Iowa City Water Company have been made. Table 1, Results of Four Valuations of the Physical Property in Use, gives some comparative reproduction values as of April 1, 1920.

In 1915 the company made application to the city council of Iowa City for increased water rates. The city council engaged the author to report upon the value of the property and the rates to which this value entitled the company. Due to the shortness of the time available for the preparation of the report, it was impossible to prepare a complete inventory of the company's property. The company submitted a statement from its books, which indicated that the value of its physical property new was $245,050.

TABLE 1

*Results of four valuations of the physical property of the Iowa City Water Company in use*

| ITEM | COMPANY'S BOOKS, JUNE 25, 1915 | REPORT BY H. M. BYLLESBY & CO., AUGUST 1, 1916 | REPORT BY J. H. DUNLAP, DECEMBER 31, 1919 (PRE-WAR COSTS) | REPORT BY J. H. DUNLAP, APRIL 1, 1920 (PRESENT COSTS) |
|---|---|---|---|---|
| Total physical property new..... | $245,050 | $332,082 | $296,002 | $669,833 |
| Depreciation..... | *22,890* | *59,299* | *43,261* | *91,695* |
| Reproduction value of physical structures..... | 222,160 | 272,783 | 252,741 | 578,138 |
| New construction less depreciation to April 1, 1920..... | *18,890* | *7,731* | | |
| Comparative reproduction values as of April 1, 1920..... | 241,050 | $279,514 | $252,741 | $578,138 |

*Note:* Operating capital and development expense or going value are omitted from tables 1, 2, and 3.

After some investigation it was concluded that this value was a fair one for use in rate-making, but the company was advised to have a complete inventory of its property made as soon as possible. Based upon the company's own estimate of the value of its property, a rate was recommended which was calculated to yield a 7 per cent net return. This rate was accepted by the company with the agreement that it would be tried out to see if the estimated return was received. The new rate was somewhat uncertain in the amount of return, since the company was about to change most of its consumers from the flat rate basis to the meter basis. It is interesting to note that the new rate brought in the estimated revenue and

provided a surplus of $2675 for the year 1916. Part of this surplus, however, was probably due to an energetic campaign for the collection of accounts unpaid in 1915.

In accordance with the recommendation that a complete inventory and valuation be made, H. M. Byllesby & Company, Chicago, were engaged for the work. Under date of August 1, 1916, a report was rendered which gave $332,082 as the value of the total physical property new. Due to the fact that the president of the Iowa City Water Company was soon called to service with his regiment on the Mexican border, no application was made by the company for an increased rate based upon the increase in value over the showing of the company's books in 1915. Following service on the Mexican border, the president of the company was called into action in the war with Germany, with the result that the rate of 1915 remained in force until 1920. In operating during 1917, 1918, and 1919 under this rate, the company lost about $22,000 over what a 7 per cent rate of return would have given. This should be looked upon as part of the contribution of the company to the citizens of Iowa City during the war period.

*The new water rates.*—In order to prevent further losses, the company engaged the author to make a valuation of its property and to suggest water rates which would yield a fair return. Accordingly he had the interesting opportunity of being in 1915 the engineer for the city council, and in 1919, the engineer for the company. The report made to the company on December 31, 1919, showed that on the basis of pre-war costs, the reproduction value of physical property new was $296,002.

Table 2 based on pre-war costs summarizes the value of the various items of property. The total depreciation of $43,261 was computed by the compound interest method with interest at 4 per cent. The depreciation obtained in this way was checked by a study of the actual condition of the property in use, and wherever necessary was changed to conform with the actual depreciation found on inspection. Subtracting the total depreciation from the pre-war reproduction cost, new, the pre-war reproduction value is found to be $252,741. This is reasonably close to the comparative reproduction value of $241,050 shown in table 1, based on the company's books, and also the comparative reproduction value of $279,514 found by H. M. Byllesby & Company.

TABLE 2

*Summary; reproduction value of physical property of the Iowa City Water Company in use, based on pre-war costs*

| DESCRIPTION | PRE-WAR REPRODUCTION COST | TOTAL DEPRECIATION | PRE-WAR REPRODUCTION VALUE | 1920 DEPRECIATION |
|---|---|---|---|---|
| Real estate | $5,000 | | $5,000 | |
| Buildings | 18,695 | $3,123 | 15,572 | $241 |
| Pumping equipment | 27,760 | 12,453 | 15,307 | 320 |
| Boiler equipment | 7,590 | 4,927 | 2,663 | 255 |
| Water supply and discharge piping | 3,973 | 1,240 | 2,733 | 91 |
| Steam piping | 1,712 | 314 | 1,398 | 35 |
| Water supply | 17,792 | 3,435 | 14,357 | 254 |
| Filter and lime room equipment | 21,772 | 1,375 | 20,397 | 222 |
| Mains | 112,742 | 3,630 | 109,112 | 273 |
| Fittings | 1,545 | 40 | 1,505 | 3 |
| Control valves | 4,177 | 943 | 3,234 | 64 |
| Meters | 20,684 | 2,960 | 17,724 | 810 |
| Hydrants | 10,429 | 3,552 | 6,877 | 256 |
| Paving over mains | 3,952 | 17 | 3,935 | 4 |
| Office furniture and fixtures | 1,480 | 222 | 1,258 | 100 |
| Tools | 2,245 | 449 | 1,796 | 450 |
| Supplies | 3,832 | | 3,832 | |
| Vehicles | 680 | 102 | 578 | 135 |
| | $266,060 | $38,782 | $227,278 | $3,513 |
| Omissions and contingencies | 11,409 | 1,751 | 9,658 | 101 |
| Legal and organization expense | 2,661 | 388 | 2,273 | 35 |
| Insurance and taxes during construction | 2,195 | 320 | 1,875 | 28 |
| Engineering | 5,056 | 760 | 4,296 | 57 |
| | $287,381 | $42,001 | $245,380 | $3,734 |
| Interest during construction | 8,621 | 1,260 | 7,361 | 112 |
| Total physical property | $296,002 | $43,261 | $252,741 | $3,846 |

The water rates in use since 1915 were as follows: Fire hydrants, $45 per annum. Flush tanks, $100 per annum. Water for municipal purposes, free.

Meter rates, payable monthly.

*Service charge*

| | per month |
|---|---|
| ⅝-inch meter | $0.50 |
| ¾-inch meter | .80 |
| 1-inch meter | 1.10 |
| 1¼-inch meter | 1.50 |
| 1½-inch meter | 2.00 |
| 2-inch meter | 3.00 |
| 3-inch meter | 5.00 |
| 4-inch meter | 10.00 |

*Output charge*

| | per 100 cubic feet |
|---|---|
| Up to and including 5000 cubic feet | $0.10 |
| For the next 45,000 cubic feet | .09 |
| For quantities in excess of 50,000 cubic feet | .06 |

The report made to the company was transmitted in duplicate to the city council, and set forth in detail the exact reasons for increased rates. As a result of this careful presentation of all the facts involved, the council of Iowa City voted unanimously to allow the company the following increased rates.

All rates to remain the same, except the output charge, which was changed to the following:

| | |
|---|---|
| Up to and including 5000 cubic feet | $0.20 |
| For the next 45,000 cubic feet | .15 |
| For quantities in excess of 50,000 cubic feet | .10 |

*Valuation based on costs as of April 1, 1920.*—Due to the fact that the company wished to issue additional stock, the author was requested to report upon the value of its physical property, based on present costs. Table 3, Summary of Reproduction Value of Physical Property in Use, based on costs as of April 1, 1920, shows that the reproduction cost, new, of the physical property of the company was $669,833. Subtracting the total depreciation, $91,695, gives a present reproduction value of $578,138. This is a value 129 per cent greater than that obtained by using pre-war costs.

In table 4 the pre-war and present costs used in the valuations are compared. It will be observed that the percentage of increase of the present costs over pre-war costs varies from 196 per cent for common red brick in place, to 39 per cent for ⅝-inch meters installed.

The great difference between the present reproduction value of the physical property of the company based on pre-war costs and

on the costs as of April 1, 1920, is an illustration of the difficulties of valuation work, using the reproduction new less depreciation method, in such general favor with the courts. It is believed that

TABLE 3

*Summary; reproduction value of physical property of the Iowa City Water Company in use, based on costs as of April 1, 1920*

| DESCRIPTION | REPRODUCTION COST | TOTAL DEPRECIATION | PRESENT REPRODUCTION VALUE | 1920 DEPRECIATION |
|---|---|---|---|---|
| Real estate | $6,250 | | $6,250 | |
| Buildings | 41,299 | $7,602 | 33,697 | $567 |
| Pumping equipment | 59,642 | 23,847 | 35,795 | 697 |
| Boiler equipment | 18,467 | 11,886 | 6,581 | 607 |
| Water supply and discharge piping | 9,068 | 2,842 | 6,226 | 209 |
| Steam piping | 3,423 | 626 | 2,797 | 72 |
| Water supply | 38,257 | 7,480 | 30,777 | 549 |
| Filter and lime room equipment | 51,038 | 3,597 | 47,441 | 502 |
| Mains | 290,086 | 9,255 | 280,831 | 696 |
| Fittings | 3,624 | 101 | 3,523 | 7 |
| Control valves | 7,616 | 1,771 | 5,845 | 119 |
| Meters | 28,448 | 4,068 | 24,380 | 1,109 |
| Hydrants | 22,127 | 7,549 | 14,578 | 534 |
| Paving over mains | 7,248 | 29 | 7,219 | 7 |
| Furniture and fixtures of office | 2,086 | 298 | 1,788 | 81 |
| Tools | 3,101 | 574 | 2,527 | 595 |
| Supplies | 4,920 | | 4,920 | |
| Vehicles | 730 | 135 | 595 | 140 |
| | $597,430 | $81,660 | $515,770 | $6,491 |
| Omissions and contingencies | 27,232 | 3,828 | 23,404 | 230 |
| Legal and organization expense | 5,974 | 816 | 5,158 | 65 |
| Insurance and taxes during construction | 4,939 | 677 | 4,262 | 54 |
| Engineering | 11,607 | 1,613 | 9,994 | 114 |
| | $647,182 | $88,594 | $558,588 | $6,954 |
| Interest during construction | 22,651 | 3,101 | 19,550 | 243 |
| Total physical property | $669,833 | $91,695 | $578,138 | $7,197 |

for rate-making purposes some quite different method of valuation will have to be worked out and standardized. In this connection it should be noted that one of the most promising of the new methods recently proposed, is that to be discussed by William G. Ray-

mond, Dean of the College of Applied Science, State University of Iowa, tomorrow morning, under the subject of "Value versus Investment as a Basis for Utility Service Rates." (See JOURNAL, January, 1921.)

TABLE 4

*Pre-war and present costs used in the valuations*

| ITEM | PRE-WAR COSTS | COSTS APRIL 1, 1920 | PER CENT INCREASE |
|---|---|---|---|
| Cast iron pipe 6-inch and over including freight and haul | $28.00 | $77.00 | 175 |
| 6-inch valves | 15.50 | 26.35 | 70 |
| 12-inch valves | 41.50 | 79.50 | 92 |
| 4-inch hydrants with two 2½-inch nozzles | 26.00 | 58.50 | 125 |
| 6-inch hydrants with steamer nozzles and two 2½-inch nozzles | 40.00 | 90.00 | 125 |
| ⅝-inch meters installed | 9.00 | 12.50 | 39 |
| Lumber in place | 40.00 | 100.00 | 150 |
| Common red brick in place | 18.00 | 53.25 | 196 |
| Concrete footings | 8.00 | 13.25 | 66 |
| Excavation | 0.50 | 1.00 | 100 |
| Cross-compound pumps per million gallon capacity | 3,800.00 | 10,000.00 | 163 |
| Boilers, 72 inches by 18 feet 0 inches, installed | 1,400.00 | 3,800.00 | 172 |
| Filters per million gallons capacity with all structures | 12,000.00 | 28,600.00 | 138 |

*Ultimate basis of value.*—It is believed that the ultimate basis of value for rate-making purposes is the value of the service performed. While the value of the service performed by a water company is difficult to estimate, yet an indication of this value is found by comparing the rates in various cities supplied with water under somewhat similar conditions.

Table 5 gives the comparative costs of water by meter rates in February, 1920, in twelve cities of Iowa with filtered supplies. It will be observed in this table that the new rates for water in Iowa City are very close to the average rates in the twelve cities. Unless there are special reasons why a local water company should exceed the average rate paid in other cities, it will be very difficult to secure a rate much beyond the average. In any case it is necessary to analyze the situation in the local community and compare it with the situation in other cities, or the local water company will earn a

bad reputation for extortion if it secures a rate much higher than its neighbors. Accordingly, whatever method of valuation may be adopted in the future, it is believed that until increased rates are established in many more cities than have as yet succeeded in obtaining them, it is going to be difficult for one company to break away very far from the average.

TABLE 5

*Yearly cost of water by meter rates in twelve cities of Iowa with filtered supplies, February, 1920*

| PLACE | POPULATION 1915 | OWNERSHIP | COST PER YEAR BY METER RATES | | | | |
|---|---|---|---|---|---|---|---|
| | | | Minimum | Gallons per day | | | |
| | | | | 50 | 100 | 150 | 200 |
| Davenport | 48,483 | Private | *$7.20 | $7.20 | $10.80 | $16.20 | $21.60 |
| | | | †9.00 | 9.00 | 12.12 | 18.18 | 24.24 |
| Cedar Rapids | 40,667 | Municipal | 10.80 | 10.80 | 10.80 | 13.56 | 17.88 |
| Council Bluffs | 31,354 | Municipal | 9.00 | 9.00 | 12.60 | 18.90 | 25.20 |
| Burlington | 24,261 | Private | 12.00 | 12.00 | 12.00 | 16.20 | 21.60 |
| Ottumwa | 22,437 | Municipal | 9.00 | 9.00 | 9.00 | 13.50 | 18.00 |
| Keokuk | 15,239 | Private | 6.00 | 6.48 | 12.96 | 19.44 | 24.84 |
| Iowa City | 12,033 | Private | *6.00 | 8.88 | 11.76 | 14.64 | 17.52 |
| | | | ‡6.00 | 10.80 | 15.60 | 20.40 | 25.20 |
| Fort Madison | 9,507 | Private | 12.00 | 12.00 | 21.60 | 32.40 | 42.30 |
| Centerville | 7,803 | Private | 6.00 | 9.00 | 18.00 | 27.00 | 36.00 |
| Creston | 7,572 | Private | 10.00 | 10.00 | 12.60 | 18.90 | 25.20 |
| Chariton | 5,235 | Municipal | 12.00 | 12.00 | 14.40 | 21.60 | 27.60 |
| Storm Lake | 3,158 | Municipal | 6.00 | 6.30 | 12.60 | 18.90 | 24.60 |
| Average of 12 cities | | | $8.83 | $9.39 | $13.26 | $19.27 | $25.20 |

* Present rate taken for average.

† Proposed rate.

‡ New rate.

*Note:* In Creston, Chariton and Storm Lake, the consumer owns the meter. In all other cases it is believed the water company owns the meter.

In other words, just so long as such facts as are contained in table 5 must be presented to city councils in Iowa, it is going to be very difficult for any privately owned water company to secure a rate of return in its capital which will enable it to attract new capital from time to time in order to make extensions and improvements. The municipally owned plants are differently situated, since by various methods of municipal accounting, they may, or

may not, be required to yield a fair rate of return on investments. It should be noted, however, that at the present time in the twelve cities listed in table 5, it is impossible to tell by looking at the cost of water, whether the plant is privately owned or municipally owned. This is an indication of the remarkable economies now practiced by privately owned water companies in comparison with municipally owned water companies.

# SELF-CLEANSING UNDERGROUND WATER COLLECTING SYSTEM[1]

By George I. Prince[2]

The author was recently called in consultation by the authorities of a western town to overcome a condition which was seriously affecting the public water supply. This supply is obtained by means of a wooden gallery, which is laid below the bed of the North Platte River, and extends under the river bed for a distance of about 600 feet from the east bank. It is of square cross-section, 36 inches by 36 inches inside dimensions, made up of 2 by 4-inch planks spaced about $\frac{5}{8}$ inches apart.

The water and the material surrounding this collecting box or gallery are of such a chemical nature as to form over and around the gallery an almost impervious curtain which tends to cut off the infiltration of the water supply. Relief has been obtained by periodically excavating the incrusted material which has formed about the gallery, but the relief is only temporary, as the cementing process again occurs within a few weeks. Another city located to the west on the same river is experiencing similar trouble due to the cementing qualities of the material of the river bed in conjunction with the hard water. The author has thought that a statement relating to this particular problem, embodying the method that is proposed to afford the relief desired and insure a constant and abundant water supply, may be of interest.

Not only has this particular gallery above referred to suffered because of the cementing qualities of the surrounding materials, but it was found on examination that considerable quantities of gravel and sand obtained entrance to the interior of the gallery and subsequently to the suction well with which it is connected, thus evidencing improper construction.

An analysis of the water supply indicated serious contamination and at the time the author was called into consultation there was a

[1]Read before the Iowa Section November 5, 1920. Discussions are invited and should be sent to the Editor.

[2]Consulting Engineer, 401 Peters Trust Building, Omaha, Neb.

number of cases of typhoid in the town. An emergency chlorinator was installed at once, with favorable results. The threatened epidemic died out. Since that time the town, on the author's recommendation, has purchased a permanent chlorinating equipment.

It is deemed essential to construct another water collecting system, to be located at a point under the river north and above a creek which discharges considerable sewage contamination into the river about a quarter of a mile above the existing gallery.

The new construction proposes a new pumping station on the east bank of the river about 250 feet east from the new water collecting system. Near this pumping station a suction well will be constructed from which a 14-inch cast-iron supply main will extend west and connect with the connecting pipe or back-bone of the water-collecting system.

The flow of the river at the location of the collecting system is from north to south. The Pathfinder Dam, constructed by the Federal Government, is located on the river something over 100 miles to the north. During the fall and winter months, this dam stores water and the stage of the water in the river below the dam is low until the irrigation season opens in the spring. It is therefore proposed to construct this collecting system during the time of low water.

The proposed collecting system will consist of a 14-inch cast-iron connecting pipe with tight joints, laid for a distance of about 900 feet in a north and south direction parallel with and within 50 feet of the east water line of the river at its low stage. The supply main extending west from the suction well, located on the east bank of the river, will connect with this connecting pipe, near its south end.

It is proposed to insert in this connecting main at intervals of 100 feet, a cast-iron special Y casting, the branch opening of which will be fitted with a flange, to which an 8-inch flange valve will be bolted.

The river bed at this particular location resembles a beach, consisting of excellent building sand underlaid with washed gravel varying in size from $\frac{1}{2}$-inch to 8-inch. The elevation of this river bed along the line of the connecting pipe varies from 83 at south end to 84 at north end, and the elevation of the flow line of the connecting pipe will vary from 74 at the south end to 75 at the north end.

Extreme low water is assumed to be 80 or 4 feet below the river bed at the south end of the collecting system. Under usual conditions the elevation of the river water during the non-irrigation season, from September 15 to March 15, will approximate 82.5.

Examinations which have been made of the sub-soil at the proposed location of the collecting system indicate conditions to exist at this point similar to what exists throughout the entire Platte River basin; namely a very large under-ground flow of water.

Attached to each of the ten 8-inch valves above referred to, there will be bolted three 12-foot lengths of 8-inch flange cast-iron pipe, which will be laid on the lines of the angle of the Y connections.

Each of these flange pipes will be drilled with three lines of one-inch holes, 50 holes in each of the pipes, equally spaced and staggered. The extreme end of each branch line will be closed by a blank flange.

Each branch line will be laid for its length of 36 feet at same elevation as its connecting Y special in a trench not less than 46 inches in width at the bottom. The trench will be excavated to a depth of at least 12 inches below the pipe and refilled with washed gravel, no piece of which will be less than 2 inches in dimension, the larger particles being placed next to the pipe. On either side of the flange pipe for a width of 18 inches and on top to a depth of 4 feet, washed gravel of like character to that below the pipe will be placed and above this the trench may be filled with excavated material.

Around each 8-inch valve there will be built a concrete manhole, the top of which will be at elevation 90 or about 2 feet above high water in the river. These manholes will not be subjected to ice-flow as the river is at its low stage during the winter months. These 8-inch valves will be built with rising stems, the floor standards and hand-wheels being erected upon the concrete covers and housed in as a protection against the weather.

In the pump house there will be installed centrifugal pumps belt-driven by oil engines. The discharge from the pumps will pass south through a 12-inch cast-iron pipe line for a distance of about 1600 feet, at which point it will connect with two existing force mains which are laid to two distributing reservoirs located on high ground east of the town.

To guard against the evils incident to the stoppage of infiltration of underground water caused by the cementing process above referred to and insure a constant and abundant supply of water, it

is proposed to employ a reverse current of water, through a by-pass pipe connecting the discharge and supply mains, under a static head of 220 feet. About once a week all of the ten 8-inch valves on the branches of the collecting system will be closed, after which each valve will be opened successively for such length of time as to allow the reverse flow from the reservoir to effectually wash out and remove all matter of a clogging nature that has accumulated around and above the branch pipe.

Assuming the pipe conditions to be such that 3000 gallons of reverse flow per minute can be delivered to each branch pipe line under a dynamic head of 70 feet, it is evident that each of the 150 holes, in the branch pipe line, would have to pass 20 gallons of water per minute requiring a velocity through the holes of 8.16 feet per second. If we assume 0.5 as the value of the coefficient of discharge, $C$, in the formula $V = C\sqrt{2GH}$ and assume $H$ to have a value of 70 feet, the formula would indicate a velocity of 32.4 feet per second. Just how much the velocity would be retarded by the gravel filling around the branch pipe lines is indeterminate, but the above figures would indicate a reduction in pressure of 75 per cent due to the surrounding gravel mass.

It is believed that the resulting velocity of the reverse flow through the surrounding gravel mass will be sufficient to thoroughly cleanse it of all the clogging matter that may have collected therein and restore normal condition.

Such is a brief statement of the problem in hand and it is hoped to obtain the opinion of those present as to the wisdom and adequacy of the proposed solution.

# FURTHER OBSERVATIONS ON THE EOSIN-METHYLENE BLUE AGAR[1]

By Max Levine[2]

At the Council Bluffs meeting of this section, in 1917, the author had the honor and pleasure to make public a simplified eosin-methylene blue agar to be employed in the confirmation of the presumptive test for B. coli in water analysis. The medium consists of 1 per cent peptone, 0.2 per cent dipotassium phosphate, 1.5 per cent agar, and 1 per cent lactose with the dyes eosin and methylene blue as indicators. The advantages claimed for the medium were, first, extreme simplicity of preparation, as neither adjustment of reaction nor filtration is required, and second, that B. coli and B. aerogenes, two organisms which have been confused in the past and which may be of different sanitary significance, are easily differentiated.

It was stated at the time that 96.9 per cent of B. coli and 82.4 per cent of B. aerogenes were correctly diagnosed from their appearance on eosin-methylene blue agar. The reliablity of this differentiation has since been checked by the author, in 1918 and 1919, at the Central Medical Department Laboratory of the American Expeditionary Forces in Dijon, France, where he found 94.2 per cent of the colonies picked as B. coli and 85.5 per cent of supposedly B. aerogenes, confirmed by further tests.

The medium has been employed with success and satisfaction in the State Water Laboratories at Iowa City; in Army Laboratory Schools for officers at Ft. Leavenworth and Yale University; at the Army Medical School, Washington, D. C.; and a number of water laboratories overseas.

In the original report, only one type of B. coli colony was described. Subsequent extensive observations have disclosed three distinct types of colony growth of this organism. The first, which is regarded

[1] Read before the Iowa Section, November 5, 1920. Discussions are desired and should be sent to the Editor.

[2] Associate Professor of Bacteriology, Iowa State College, Ames, Iowa.

as the typical B. coli and which constitutes 95 to 98 per cent of the types encountered, has the following characteristics:

Isolated colonies are 2 to 3 mm. in diameter and they show very little tendency to run together. The colonies are slightly raised, the surface is flat or slightly concave, and the edge is even and regular when viewed with the naked eye. By transmitted light, they appear dark, and have almost black centers which extend more than three-fourths of the distance across the diameter of the colony. By reflected light they appear dark, button-like, often concentrically ringed and show a distinct, characteristic, greenish, metallic sheen.

*Types of colonies of B. coli on eosin-methylene blue agar*

| SIZE | I. WELL ISOLATED COLONIES 2–3 MM. IN DIAMETER | II. SAME AS I | III. LARGER THAN I USUALLY 3–4 MM. DIAMETER |
|---|---|---|---|
| Confluence | Neighboring colonies do not run together | Same as I. | Same as I |
| Elevation | Slightly raised | Slightly raised | Flat,—effuse. Appears almost as if part of the surface of medium |
| Surface | Flat or slightly concave | Usually slightly convex | Flat |
| Edge (viewed with naked eye) | Regular, entire | Same as I | Irregular, crenated, or lacerated |
| Appearance by transmitted light | Dark, almost black centers which extend three-fourths across the diameter of colony, surrounded by a narrow lighter zone | Purple centers | Like I but outer zone is irregular and often much broader |
| Appearance by reflected light | Dark button-like, often concentrically ringed, with a greenish metallic sheen | Wine colored, metallic sheen not present | Distinct greenish metallic sheen like I |

A second type of B. coli colony encountered resembles the typical form just described with respect to size and elevation, but there is no metallic sheen, the colonies being wine-colored.

The third variety of colon colony appearing on this medium is somewhat larger (3 to 4 mm.) than the typical form described. It grows effusely, or flat, the edge is crenated or irregular, and the central portion has a distinct metallic sheen.

Several varieties of B. aerogenes have also been observed. The most frequently encountered colony type and the one which was originally decribed is as follows:

Well isolated colonies are larger than B. coli, usually 3 to 5 mm. in diameter, and neighboring colonies will quickly run together forming a limy mucoid-appearing mass. The colonies are moderately raised, markedly convex, while the centers are a light to a dark brown and relatively small, less than one-half the diameter of the colony. By reflected light, they appear much lighter than B. coli and there is no metallic sheen.

The second type of B. aerogenes encountered produces colonies somewhat larger than the one described above. The center is very dark, wine-colored, and relatively large (often being more than three-fourths of the diameter of the colony) and there is a glossy metallic sheen in young cultures. Confluence is very marked.

A third type of B. aerogenes observed is a small colony, often smaller than B. coli. The colonies are raised but the center drops precipitately like a crater. By reflected light, the small dark central portion may show a metallic sheen.

Whether these different types of colonies are of different sanitary significance has not as yet been determined but it is apparent that the medium affords a means of differentiating varieties of B. aerogenes and B. coli which the author believes has not been attained with any of the other media employed for confirming the presumptive test. It has also been observed that B. cloacae and particularly Ps. pyocyaneus, which have occasionally been mistaken for B. coli on the standard Endo medium, grow very poorly on eosin-methylene blue agar, forming colonies which are not likely to be mistaken for either B. coli or B. aerogenes.

Test tubes may be substituted for petri dishes if desired. The value of such a change is, first, reduction of expense, as only 3 or 4 cc. of medium is needed for each test tube, while about 15 cc. is usually employed with a petri dish; and, second, test tubes may be

*Types of colonies of B. aerogenes on eosin-methylene blue agar*

| SIZE | I. WELL ISOLATED COLONIES 3–5 MM. IN DIAMETER | II. SLIGHTLY LARGER THAN I. 3–6 MM. IN DIAMETER | III. SMALLER THAN I. 1½–3 MM. DIAMETER |
|---|---|---|---|
| Confluence | Neighboring colonies run together or fuse | Like I | Colonies remain isolated and distinct, little tendency to fuse |
| Elevation | Moderate raised | Considerably raised | Slightly raised |
| Surface | Convex | Convex | Center drops precipitately like crater |
| Edge (naked eye) | Regular | Regular | Regular |
| Appearance by transmitted light | Light to dark brown centers which are relatively small (less than one-half diameter of the colony) surrounded by a broad grey or pink zone | Very dark, large wine colored centers often three-fourths diameter of colony | Small pinhead or slightly larger center, light to a dark brown |
| Appearance by reflected light | Metallic sheen not present. Colonies appear glossy and mucoid | Like I but very strong green metallic sheen, particularly in young cultures | Colonies distinct, comparatively dull and the crateriform center may have a metallic luster |

stored for long periods, whereas the medium in petri dishes would have to be prepared at intervals of a week or less. The method of preparing test tubes is briefly as follows:

To 100 cc. of sterile melted phosphate agar is added 5 cc. of a 20 per cent lactose, 2 cc. of 0.5 per cent aqueous methylene blue solution and 2 cc. of a 2 per cent aqueous eosin. The medium is then poured aseptically into sterile test tubes and allowed to solidify so as to give a long slant without a butt. To confirm the presumptive

test, inoculation is made on the surface of these slants with a straight platinum needle. If desired the medium may be placed in test tubes and sterilized in the Arnold or in the autoclave in the ordinary manner. In that case it will be found that the medium, when it comes out of the sterilizer, is fluorescent like eosin; but as it cools, the typical wine color returns. The author prefers to prepare tubes as described above.

The author has felt for sometime that before much light will be thrown on the true relative incidence of B. coli and B. aerogenes in water, sewage, etc., it will be necessary to first modify our preliminary enrichment media or other conditions so as to enable an investigator to isolate or suppress B. coli or B. aerogenes at will.

*Vigor of growth of B. coli and B. aerogenes on eosin-methylene blue agar with various concentrations of brilliant green. (Observations after twenty-four hours)*

| CULTURE | CONCENTRATION OF BRILLIANT GREEN | | | | |
|---|---|---|---|---|---|
| | None (control) | 1:400,000 | 1:300,000 | 1:200,000 | 1:100,000 |
| B. aerogenes 8b | +++ | +++ | +++ | +++ | ++ |
| B. aerogenes 11 | +++ | +++ | ++ | + | + |
| B. aerogenes 13 | +++ | +++ | +++ | + | + |
| B. aerogenes 17d | +++ | +++ | ++ | ++ | + |
| B. coli 19b | +++ | − | − | − | − |
| B. coli 20b | +++ | − | − | − | − |
| B. coli 21c | +++ | + (Faint atypical) | + (Faint atypical) | − | − |
| B. coli 22 | +++ | + (Faint atypical) | + (Faint atypical) | − | − |

Attempts to devise media which will inhibit B. aerogenes and at the same time permit B. coli to grow have not been very successful. By the addition of 1:300,000 to 1:400,000 brilliant green to eosin-methylene blue agar, the growth of B. coli may be markedly suppressed; whereas B. aerogenes grows very luxuriantly.

The researches of Rogers and his associates in the Bureau of Animal Industry, United States Department of Agriculture, and those of the Iowa Engineering Experiment Station, which have been adequately confirmed by many investigators, have shown that B.

coli is characteristic of the intestinal tract of man and the higher animals; whereas B. aerogenes, which has been formerly confused with B. coli, is very rarely encountered in human or animal excreta but constitutes the predominating type in soil and on grains. These two organisms, it is believed, should not be considered of the same sanitary significance, and water works men are naturally interested in obtaining information as to the relative incidences of these two bacteria in raw and treated water. The eosin-methylene blue agar is suggested as a simple and reliable medium for the detection of B. coli and at the same time differentiating it from its less objectionable relative, the B. aerogenes.

# TROPICAL WATER SUPPLIES[1]

By Don M. Griswold[2]

The water supply of the islands of the American tropics presents as varied a picture as could be well imagined. The islands compose a chain which sweeps in a wide semi-circle from the tip of Florida to within sight of the Venezuelan coast. The larger islands on the western end of the semi-circle are Cuba, Haiti, Santo Domingo and Porto Rico. The remainder of the islands in the American tropics are known as the Lesser Antilles. They are smaller, less highly developed and remain in a sanitary condition not far advanced from that in which the early explorers found them on the discovery of the American continent.

The islands are all under the sovereignty of the United States, France and Great Britain. The recent purchase by the United States of the Island of St. Thomas from Denmark will give us a navy base from which our fleet, in conjunction with those of the British and French, can "lock up the Caribbean Sea."

Trinidad is the southernmost island of the chain and also the one which can be seen from the mainland of the South American continent.

Down the middle, for almost the whole length of the island, runs a very rugged range of mountains. No one is reported to have ever crossed this range and indeed it is quite unnecessary, for a little 30-foot steamboat makes the circuit of the island, picking up and discharging freight at six points every 24 hours. The impassibility of this mountain fastness makes it an ideal place for a catchment area and it is so utilized. The amount of water and the number of streams are not as great as one would be led to suspect by the great size of the catchment area and the further fact that the mountains are comparatively high and densely covered with impenetrable tropical vegetation. It was, however, from these mountain streams that the pirates

[1] Read before the Iowa Section, November 5, 1920. Discussions are invited and should be sent to the Editor.

[2] Associate Professor of Preventive Medicine and Hygiene; State University of Iowa; State Epidemiologist of Iowa; Iowa City, Iowa.

of old landed to replenish their water butts, for this was one of the most famous watering places along the Spanish main.

The chief city on the island is Port-of-Spain. The municipal water supply is caught on the two sides of the Maraval Valley, and collected in a pond or lake some five or six miles from the city. This valley is uninhabited except by animals. It is covered with dense tropical jungles which make it impassable for human beings or large animals. This water is of good sanitary quality at all times.

About six miles south of the city is the St. Joseph River which runs from the mountain to the sea a distance of about six miles. It was up this river that Sir Walter Raleigh said he sailed his boats to the City of Eldorado, whose streets were paved with gold. It was near Eldorado where he found the pit of iron pyrites with which he loaded seven of his ships and sailed for England, which shortage of chemical knowledge has made him and his "fool's gold" laughing stock ever since.

Before the river leaves the fastness of the jungle, its sanitary quality is very good, but from the time it strikes the flatland it is used by Mohammedans for bathing and is usually not fit to drink. At the author's solicitation the warden of Tacarigua forbid bathing in the first half mile of the stream after it leaves the mountain, so that the natives might have a clean source of water supply.

The next island is Tobago, made famous by the fact that Alexander Selkirk was cast ashore there and while there accumulated the experiences forming the basis upon which Defoe afterward wrote the story of Robinson Crusoe. Tobago is a low flat island with two small streams which do very little more than furnish water for man and animals. During the dry season, sailing vessels are frequently sent to Trinidad to bring water for the stock, a very expensive procedure.

The island of Dominica is the one example of an island with a liberal water supply. This is the island that produces 90 per cent of the world's supply of limes. There are said to be 365 rivers on the island, which has an area of only 200 square miles. The author tried to count them on several occasons but became hopelessly lost in the task. The whole island is one vast mountain range, the only village being on the seacoast on a small piece of flat land. The multitudinous rivers take their origin at unknown places back in the mountain jungles and run but a short distance to the sea. Probably none of them is more than five miles long, as the island is 10 miles wide by 20 miles long.

White men going to the tropics are warned of the many polluted water supplies and are advised to drink nothing but boiled water. The author considers this good advice but on the island of Dominica one can find good water to drink that has not been boiled. It is one of the few places in the tropics of the old world or new where it is safe to drink unboiled water. One of the reasons for it not being polluted is that the natives are transplanted Africans, who are not at all addicted to the use of water for bathing purposes.

Mohammed incorporated many wise sanitary precepts in his code of morality and his followers are taught that their morning prayer will be heard much better if they are bathing at the time the rising sun first shines upon them. It is an interesting sight to see the Mohammedans wading out into the river to bathe and all saying their prayers as the first rays of the sun break over the mountain top.

The natives of Dominica are the Caribs, for whom the Caribbean Sea was named. They are the cousins of our North American Indians. High cheek bones, thin noses, straight black hair, copper colored skin and many other Indian characteristics remind one of the Indians of this country.

The last island to which attention is drawn is Antigua. This is a low, flat, almost treeless island with only one small mountain 1000 feet high. There is not a single pond, lake or stream of fresh water on the island that lasts the year through. Certain places on the island are so near the sea level that the salt water of the ocean appears in the fields of sugar cane and makes barren places which are known as "gall spots;" one especially large one is known as "the dead sands." These places occur because the salt content of the ocean water prevents the germination of seeds and prevents the growth of all but a few plants. To sink a well would, of course, be folly for the underlying water is salt.

Aside from a small catchment area built by the British government about 200 years ago the sole supply of drinking water is caught from roofs. Every government official has his home completely guttered and every drop of rain it is possible to collect is delivered to a cistern. These cisterns are built very substantially of cement and always entirely above ground, which is an improvement over American practice for cisterns of household size. The roofs of churches and public buildings are also all completely guttered, and the water saved for the emergencies that are almost sure to arise.

In this country "crime waves" seem to have some relation to cold weather, lack of food and unemployment. In the tropics there is no cold weather, food can be had for the picking of it and no native works while he has a penny in his pocket. So that a breakdown in the public morale is not likely for causes operative here. However, the British and French governments have found by bitter experience that a native population will rise up and fight for water. Man can live 60 to 70 days without food. Several men have abstained from food for 40 days and recovered health. Death usually comes on the fifth to seventh day of abstinence from water. Men are driven into a fighting fury by absence of food or water in spaces of time much shorter than these. So the governments try to keep the native population supplied with water during the dry season.

On studying the statistics for a period of years, it is found that the cause of death listed as "Diarrhoea and Enteritis under 2 years of age" accounts for 10 to 15 per cent of deaths from all causes. The infant death rate is appallingly high, but even at its dizzy height it is exceeded by the birth rate, 70 per cent of which are illegitimate under the British law.

In conclusion it should be said that each of these islands presents problems of water supply as widely varied as one can imagine. The variety of water-supply problems presented to the sanitarian is almost unlimited and more than entrancing. The American tropics are, without doubt, the garden spot of the world in everything except sanitation. If its physical sanitation were such that it was a safe place in which to live we would all be living there. In the progress of the world's enlightenment, sanitary knowledge slipped lightly by this beautiful spot, but the great work begun by Gorgas at Panama is having its effect and evidences of an awakened interest can already be seen.

# PIPE CASTING AT PONT-A-MOUSSON[1]

By George C. Whipple[2]

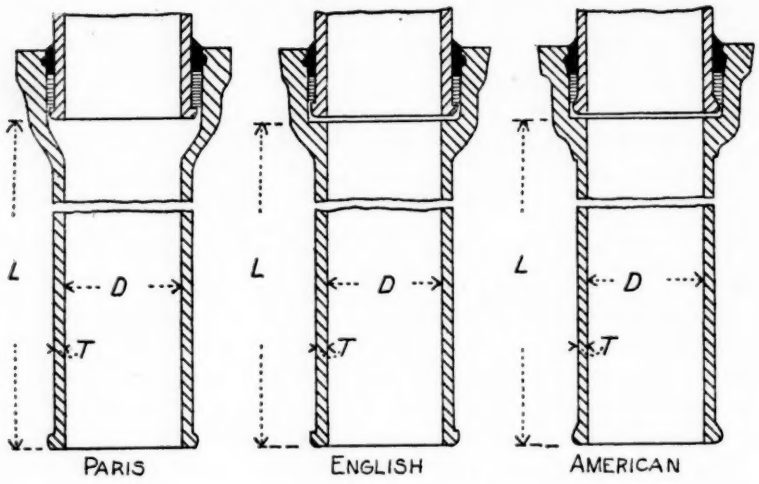
In April, 1920, during a visit to M. Ed. Imbeaux, Ingenieur de Ponts et Chaussées at Nancy, France, the speaker had the pleasure of visiting the pipe foundry of the Société Anonyme des Hauts-Fourneaux et Fonderies at Pont-a-Mousson, France. As this is one of the two most important pipe foundries in France and as it went through an interesting experience during the war, perhaps a few notes in regard to the visit may be worth placing on record.

Pont-a-Mousson is a small town north of Nancy on the Moselle River. It is in the region of the iron mines. The foundry was established in 1856. In 1879 the works covered 435 acres and employed over 1500 men. Just before the war the plant covered 4200 acres and had 6000 employees. It included five blast furnaces, twelve cupolas for melting the iron, foundry buildings which covered ten acres, two large shops, a power station and the dock on the river nearly half a mile long. In 1914 it had a capacity of 300,000 tons of castings a year, of which 150,000 tons represented cast iron pipe. The company also manufactured slag brick and cement. Its iron mines had an annual capacity of over a million tones of ore. The company also operates another large foundry at Foug, near Toul.

Most of the cast iron pipe manufactured is known as "universal" or "City of Paris" pattern, but pipes of English, Swedish, and German patterns are also made. All pipes are coated with coal tar or Angus Smith coating and are tested under a pressure of 500 to 800 feet of water as required before coating. The French pipe differs somewhat from the American pipe. In general it is thinner and has heavier bells. The listed weights for light and heavy pipe correspond approximately to Classes A and B of the American Water Works specifications. All pipes have uniform outside diameters so that the thicker pipes have slightly less capacity than the

[1] Revised from an informal talk before the New York Section on November 17, 1920. Discussion is invited and should be sent to the Editor.

[2] Professor of Sanitary Engineering, Harvard University, Cambridge, Mass.




*Standard dimensions of cast iron pipe (American, English, French)*

| CITY OF PARIS | | | ENGLISH | | | | | AMERICAN (A. W. W. A.) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| D | L | Weight | D | L | Class | T | Weighs | D | Class | T | Weight |
| *ins.* | *ft. ins.* | *lbs./ft.* | *ins.* | *ft. ins.* | | *ins.* | *lbs/.ft.* | *ins.* | (L=12′) | *ins.* | *lbs./ft.* |
| 3.94 | 9 10 | 17 | 4 | 9 10 | L | 0.36 | 17 | 4 | A | 0.42 | 20 |
| | | | 4 | | H | 0.38 | 18 | 4 | B | 0.45 | 22 |
| 5.91 | | 27 | 6 | | L | 0.39 | 27 | 6 | A | 0.44 | 31 |
| | | | 6 | | H | 0.50 | 36 | 6 | B | 0.48 | 33 |
| 11.81 | 13 1½ | 65 | 12 | 13 1½ | L | 0.51 | 65 | 12 | A | 0.54 | 73 |
| | | | 12 | | H | 0.69 | 93 | 12 | B | 0.62 | 82 |
| 17.72 | | 114 | 18 | | L | 0.61 | 114 | 18 | A | 0.64 | 129 |
| | | | 18 | | H | 0.75 | 156 | 18 | B | 0.75 | 150 |
| 23.62 | | 168 | 24 | | L | 0.69 | 178 | 24 | A | 0.76 | 204 |
| | | | 24 | | H | 0.8° | 240 | 24 | B | 0.89 | 233 |
| 29.53 | | 245 | 30 | | L | 0.81 | 262 | 30 | A | 0.88 | 292 |
| | | | 30 | | H | 1.00 | 342 | 30 | B | 1.03 | 333 |
| 35.42 | | 314 | 36 | | L | 0.91 | 352 | 36 | A | 0.99 | 392 |
| | | | 36 | | H | 1.19 | 482 | 36 | B | 1.15 | 454 |
| 43.31 | | 453 | 42 | | L | 1.00 | 463 | 42 | A | 1.10 | 513 |
| | | | 42 | | H | 1.25 | 559 | 42 | B | 1.28 | 592 |
| 49.22 | | 575 | 48 | | L | 1.11 | 581 | 48 | A | 1.26 | 667 |
| | | | 48 | | H | 1.38 | 739 | 48 | B | 1.42 | 750 |

Note: Values for French and English Pipe from Catalogue of the Pont-A-Mousson Blast-Furnaces and Iron Foundries Co.

thinner ones. The accompanying diagram and table illustrate this pipe.

The company also manufactures a cast iron pipe surrounded by steel hoops for high pressures. It also manufactures gas pipe and soil pipe. Besides its European trade the company does a large export business. The speaker learned in Switzerland, however, that Pont-a-Mousson pipe was not much in use there, as it was possible to obtain at other places a metal which contains less sulphur and phosphorus and is, therefore, less brittle.

Pont-a-Mousson is said to have been the first French city to be bombarded by the Germans. On July 31, 1914, the works were shut down on account of mobilization. Bombardment began on August 11, and from September 5 to 11 the place was occupied by the enemy. After the Germans had been driven out, however, bombardments continued throughout the war until the armistice. During this time, 5000 bombs and shells fell on the works. Later, the buildings were badly damaged. Many of them were utterly destroyed. Extensive defensive measures had been made, however, sand bags being piled around the machinery, and in many cases these served the purpose of preventing the machinery from being destroyed. In 1915 the company attempted to resume work. This being unsuccessful, however, the company transferred its activities to its branch plan at Foug, and although this plant was only twelve miles from the front and was continually subject to air raids, work was continued for 52 months and during this time, 5,000,000 shells, weighing in the aggregate over 200,000 tons, were manufactured.

After the war operations were resumed as rapidly as possible. Mining was begun November 11, 1918; one cupola was put in operation in April, 1919; pipe casting began in August; and one of the blast furnaces was started in the following November. At the time of our visit repairs were still in progress.

We had an interesting interview with the president of the company, M. Camille Cavallier. He is a very efficient administrative officer and a man of broad education, interested not only in his foundry work but in the welfare of his operatives and in all of the great social and economic questions of the day. He had much to say in regard to the labor situation and believes that France cannot prosper unless the laborers are willing to work nine or ten hours a day instead of eight hours, which they desire. The following quotation from an address which he made before the Association Gen-

erale des Ingenieurs, Architects, et Hygienistes Municipaux, at Naney in 1909, will illustrate the interest which M. Cavallier has in these questions.

We are in a country which for half a century has had an extraordinary industrial development, but it is a curious thing that while health is the most important human possession, man gives more care to his material interests than to his health. When a person is sick, he consults a physician, but no one ever thinks of consulting a hygienist on measures to prevent disease. The profession of "Consulting Doctor of Hygiene" does not exist.

Demographers deplore France's low birth-rate; journalists, generally celibate themselves, denounce the peril; senators preach a crusade of repopulation. Vain efforts! France in all things is economical. Population decrease, moreover, is concomitant with civilization. But while births are few, infant deaths are many. It is a very grave situation for France. Infant mortality figures in other countries show that it is possible to save these lives. The future of France is not in the fathers of families, but depends upon engineers, architects and municipal hygienists. It is these who will increase the population of France by saving lives. An engineer of a city of 25,000, who by his work reduces the death-rate from 20 to 14 per 1000 saves 150 lives a year, and is really increasing the population as much as 3000 fathers of families will do. M. Imbeaux has already done for Nancy as much as 9000 fathers of families, and still looks happy.

The brilliant court of Louis XIV in the 17th century was the admiration of the world, but the sanitary conditions were such that they cannot be described. Louis XIV brought the waters of Marly to Versailles, but it was not for bathing or other sanitary use, but for embellishing the park. One is stupefied, on visiting Versailles, to contrast the wonderful majesty in which the sovereigns held themselves, with the apartments in which they lived, so badly ventilated, so insanitary, so unclean. Times have changed. France has sanitary laws, but as Montesquieu has said, "Laws do not change customs." What the state cannot obtain by law, citizens can themselves do.

The basis of public hygiene and individual hygiene is water,—clean, pure, wholesome, and abundant. In our country we have no lack of water. The reason we don't have better water supplies is because of the powerlessness, the carelessness, the lack of courage of the city electors, the lack of public opinion.

# REPAIRS TO A FLEXIBLE-JOINTED PIPE UNDER THE EAST RIVER TO BLACKWELLS ISLAND[1]

By George E. Rodman[2]

Blackwells Island is supplied with water from Manhattan by a 12-inch flexible-jointed pipe about 1000 feet long laid in 1908 under the East River from the foot of East 68th Street, and by a 12-inch steel pipe over the Queensboro Bridge, laid in 1917. Complaint was received on September 20, 1919, of lack of pressure on the Island, it having dropped from 39 to 15 pounds. Investigation showed leakage at the rate of about 3,000,000 gallons in 24 hours on the line under the river and this line was shut off.

The services of a diver were obtained on September 21, and an air compressor was connected to the line so that the escaping air showing in the river would determine the location of the leak. The capacity of the compressor was 24 cubic feet of free air per minute, and it could maintain a pressure of 125 pounds. It was only used under about 20 pounds on this job.

As the air was escaping from the line about 50 feet from the shore, the diver cleaned off the pipe at the bulkhead line and followed it out to where the air was escaping, it being necessary to remove considerable mud and some stones from over the pipe, which was in about 45 feet of water at this point. Owing to the current being about 5 feet a second, the diver could only work on slack water, working for periods of about an hour each on two tides a day. The mud was from 3 to 4 feet deep over the pipe and was washed away by a water jet supplied from a fire hydrant on shore, 2½-inch hose being used from the hydrant to the launch and 2-inch hose ending with a piece of 2-inch iron pipe about 4 feet long from the launch to the river bottom.

Upon uncovering the pipe it was found that a split sleeve had been placed over the joint that was leaking, and that the bolts and

[1]Read before the New York Section, December 11, 1919. Discussions are invited and should be sent to the Editor.

[2]Assistant Engineer, Department of Water Supply, Gas and Electricity, Municipal Building, New York, N. Y.

washers holding the split sleeve together had rusted away so that the sleeve was partly off the pipe. The gaskets between the two halves of the sleeve had been made of what appeared to be several thicknesses of rubber fiber hose, and this was badly disintegrated. The joints on the ends of the sleeve were originally made with lead wool, but most of this had been washed away.

The upper half of the sleeve was raised to the surface, cleaned off and replaced over the joint. It was necessary to use several $\frac{1}{8}$-inch lead gaskets between the halves, as the angle formed by the two lengths of pipe would not permit them to meet and leave a space for the lead calking at the ends.

It was at first proposed to use a nickel alloy for the bolts and washers on this sleeve, but the department was advised that when it was used under water in connection with iron the results might be unsatisfactory. A bronze was accordingly used for the 48 $\frac{1}{2}$-inch by $3\frac{1}{2}$-inch bolts and 96 washers which cost $15.60. This use of bronze is more or less experimental, for information concerning the best material for bolts for such sleeves, submerged in salt water, is very scanty.

Although the bolt holes in the flanges of the sleeve were $\frac{7}{8}$-inch, the original bolts were but $\frac{3}{8}$-inch and the diver asked that the new ones be not larger than $\frac{1}{2}$-inch on account of the difficulty in making the two sets of holes meet under water.

After the sleeve was bolted in place, the space at the ends was calked with lead wool. A meter test showed that the leakage on the entire line was only at the rate of 36,000 gallons in 24 hours, so the line was put in service on October 6.

The cost of the repair was $1,388.60. If bolts and washers of a non-rusting or non-corroding metal had been used originally, it is probable this repair job would never have been necessary. It is suggested, therefore, that any member who has had an opportunity to observe the life of steel or alloy bolts on iron or steel parts submerged in salt water will do a real service to engineers by sending to the Editor of the JOURNAL a statement of his observations.

# THE DEPLETION OF GROUND-WATER SUPPLIES[1]

By Robert E. Horton[2]

*Time required for a well to reach equilibrium with constant draught.* It not infrequently occurs where wells are used for public water supply or for irrigation that there is a progressive depression of the water table, sometimes continuing for years. It is evident that a new well might derive an initially large but ever decreasing supply from connate water, even with no infiltration. In general, with constant draught and infiltration, the well will reach equilibrium when

Total inflow + Storage draught = Total draught.

For a horizontal or nearly horizontal water table, the inflow takes place by infiltration over the cone of depression created by the well. If the slope of the cone of influence is known, then the radius and volume of the cone for any depth of depletion, and the area over which infiltration supples the well at any moment can be determined. Miller-Brownlie[3] has attacked this problem at a different angle from other investigators. Instead of assuming, as is often done, a constant maximum radius of the cone of influence, he assumes a minimum constant slope of the water table within the limits of the cone, which slope he assumes will be quickly reached after pumping begins and will remain constant as depression advances. He says:

> Observations made during the past few years of the subsoils of the Punjab have shown that the slopes necessary to cause water motion have varied from 1 in 260 in moderately coarse sand to 1 in 175 for fairly fine sand. In gradients flatter than this in each type of sand there is no apparent motion. Capillary attraction interferes with the true flow, and investigation into the forward motion becomes greatly involved. Observations indicate that any lateral or forward motion of water, where the hydraulic gradients are slightly less than those mentioned, is so slow that for practical purposes it may be neglected, the actual velocity probably not exceeding a few inches per day.

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[1] Discussions of this paper are requested and should be sent to the Editor.

[2] Consulting Hydraulic Engineer, Voorheesville, N. Y.

[3] "Subsoil Water in Relation to Tube Wells for Irrigation," *Engineering and Contracting*, March 31, 1920, page 355.

In effect this amounts to the assumption that there is a minimum angle of friction for ground water similar to the angle of friction for solids. If the slope of the water table is at less than this minimum angle, motion, or at least appreciable motion, in supplying a well, will not occur.

Miller-Brownlie applies this principle to the determination of the progressive depletion of wells, making computations by yearly intervals.

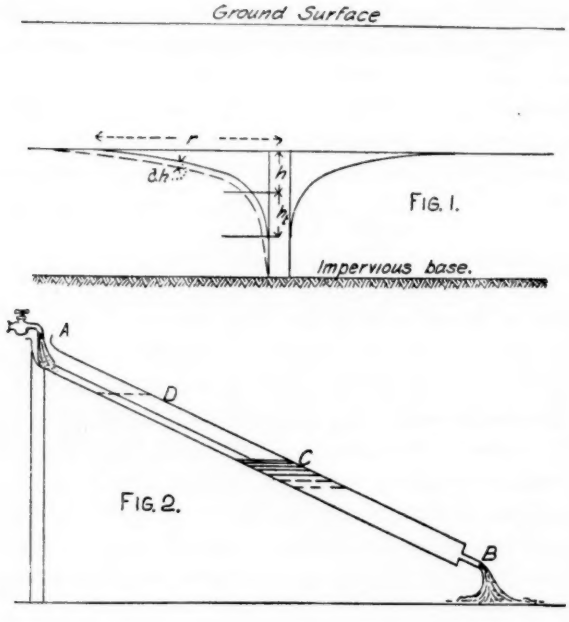


FIG. 1. CONDITIONS ADJACENT TO A PUMPED WELL

FIG. 2. SHRINKAGE OF ARTESIAN AREA WITH INCREASED DRAUGHT OF WATER

The same results may be obtained by the use of a general formula derived as follows: Referring to figure 1, let

$r$ = radius of cone of influence at time $t$ in years;
$m$ = assumed minimum slope for the given soil;
$h$ = the true or cone depression of the water table at the well, due to the slope of the water table.

This is designated by Miller-Brownlie as the "vertex head." This does not include the abrupt depression $h_t$ due to entrance resistance

immediately around the well, which may amount to a few feet. This is designated by Miller-Brownlie as the "trumpet head," from the resemblance of the form of depression at the well to the mouth of a trumpet.

$h = mr$ and
$H = h + h_t$ = total depression.
$P$ = available porosity of the medium.
$I$ = Infiltration depth inches on surface per year.
$F$ = Infiltration rate at time $t$, cubic feet per year.
$Q$ = Rate of draught or pumpage, cubic feet per year.
$q_g$ = Rate of draught or pumpage, millions of gallons per 24 hours.
$S$ = Total storage depletion in cubic feet at time $t$.

Then in units of cubic feet per year:

$$S = \frac{\pi P}{3} m^2 r^3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$F = \frac{I}{12} \pi r^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The limit of the cone of influence for constant pumping will be reached when $F = Q$, or when

$$r = \sqrt{\frac{12\,Q}{\pi I}} \text{ or } h_c = m\sqrt{\frac{12\,Q}{\pi I}} \ldots\ldots\ldots\ldots (3)$$

To determine the time when this limit will be reached, or when any less depletion will be reached, it is necessary to solve the storage equation

$$\text{Inflow} = \text{Outflow} + \text{Gain or} - \text{Loss of storage} \ldots\ldots (4)$$

For an increment of time $dt$, during depletion, the loss of storage is the difference of volume of porosity in two cones of altitudes $h$ and $h + dh$ respectively, and side slopes $m$. This is very approximately equal to the convex surface of a cone of base radius $r$ and altitude $h$, or $\pi r^2$ sec. $A$ multiplied by $P \cos A\, dh$ or

$$dS = P\pi r^2 dh \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Substituting (2) and (5) in (4) for time $dt$,

$$\frac{I}{12} \pi r^2 dt = Q dt - P\pi r^2 dh \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

But $r = h/m$

$$\text{Let } A = \frac{I\pi}{12\,m^2} \cdot \qquad B = \frac{P\pi}{m^2} \quad \ldots\ldots\ldots\ldots\ldots (7)$$

$$Ah^2\mathrm{d}t = Q\mathrm{d}t - Bh^2\mathrm{d}h \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Divide by d$t$ and transpose —

$$\mathrm{d}t = \frac{Bh^2}{Q - Ah^2}\,\mathrm{d}h \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

This can be integrated by known forms, giving

$$t = 2.30\,\frac{BQ}{2\,A\sqrt{AQ}}\,\log_{10}\frac{\sqrt{Q} + h\sqrt{A}}{\sqrt{Q} - h\sqrt{A}} - \frac{Bh}{A} \quad \ldots\ldots (10)$$

Miller-Brownlie's treatment of this subject, and the author's formula based thereon, differ from the usual mode of treatment in that it is assumed that the supply of the well is ultimately derived entirely from infiltration over the area covered by the cone of influence, whereas in the usual method of treatment heretofore it is assumed that the well is supplied by water flowing into the cone of influence around the periphery of this cone, and infiltration on the area of influence is neglected. The usual formulas give the yield of the well in terms of the depth of depression of the water table. The author's formula not only gives the ultimate depth of depression which is approached as a limit as pumping continues, but it will also give the depression of the water table at the well at any chosen time interval after pumping begins.

Inspection of the formula shows that the value of $t$ becomes infinity when the denominator of the logarithmic term becomes zero, that is when

$$\sqrt{Q} = h\sqrt{A} \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

or when

$$h = \sqrt{\frac{Q}{A}} \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

This is the limit of the vertex head, and this formula gives the same result as that obtained by the solution of equation (3). In the example given herewith, the limiting depression approaches 33.68 feet. Table 1 contains the results of a solution of equation (10) for a well used as an example to illustrate the method by Miller-Brownlie. The results agree substantially with those obtained by Miller-Brownlie by solution for successive finite increments of $h$.

TABLE 1

*Calculation of Depletion of Ground Water in a Well for $Q = 3$ c.f.s. $I = 0.8$ ft. per Year. $M = 0.00555$ and $P = 0.30$ where*

$$t = 14.54 \text{ Log.}_{10} \frac{9\ 592 + 285h}{9592 - 285\ h} - 0.375\ h.$$

| $h_c$ | $285\ h_c$ | $9592 + 285\ h_c$ | $9592 - 285\ h_c$ | COL. (3) ÷ COL. (4) | LOG. COL. (5) | 14.54 × COL. (6) | $0.375\ h_c$ | $t$ = COL. (7) − COL. (8). YEARS |
|---|---|---|---|---|---|---|---|---|
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) |
| 5 | 1,425 | 11,017 | 8,167 | 1.348 | 0.12969 | 1.88 | 1.87 | 0.01 |
| 10 | 2,850 | 12,442 | 6,742 | 1.845 | 0.26600 | 4.10 | 3.75 | 0.35 |
| 15 | 4,275 | 13,867 | 5,317 | 2.608 | 0.41631 | 6.03 | 5.62 | 0.41 |
| 20 | 5,700 | 15,292 | 3,892 | 3.928 | 0.59417 | 8.61 | 7.50 | 1.11 |
| 25 | 7,125 | 16,717 | 2,467 | 6.78 | 0.83123 | 12.04 | 9.38 | 2.66 |
| 28 | 7,980 | 17,572 | 1,612 | 10.89 | 1.03703 | 15.08 | 10.50 | 4.58 |
| 30 | 8,550 | 18,142 | 1,042 | 17.42 | 1.24080 | 17.98 | 11.25 | 6.73 |
| 31 | 8,835 | 18,427 | 757 | 24.35 | 1.38650 | 20.16 | 11.62 | 8.54 |
| 32 | 9,120 | 18,712 | 472 | 39.65 | 1.59824 | 23.20 | 12.00 | 11.20 |
| 33 | 9,405 | 18,997 | 187 | 101.50 | 2.00647 | 29.08 | 12.38 | 16.70 |
| 33.5 | 9,547.5 | 19,140 | 45.5 | 421.00 | 2.62428 | 38.04 | 12.56 | 25.48 |

Let $Q = 3$ c. f. s. $= 92{,}016{,}000$ cu. ft. per year.

$I/12 = 0.8.$ $\quad P = 0.30.$ $\quad m = \frac{1}{180} = 0.00555.$

$$\frac{1}{m} = 180. \qquad \frac{1}{m^2} = 32{,}400.$$

$$A = 0.8 \times \pi \times 32{,}400 = 81{,}389.$$

$$B = 0.3 \times \pi \times 32{,}400 = 30{,}521.$$

$$B/A = 3/8 = 0.375.$$

$$\sqrt{Q} = 9591.7.$$

$$\sqrt{A} = 285.$$

$$\sqrt{AQ} = 2{,}733{,}720.$$

$$2\sqrt{AQ} = 5{,}467{,}440.$$

$$Q/2\sqrt{AQ} = 16.83.$$

$$t = 14.54 \log_{10} \frac{9592 + 285h}{9592 - 285h} - 0.375\ h.$$

In this example a depletion of nearly 20 feet occurs in the first year because the infiltration area is naturally small, and the supply

is at first drawn mainly from ground-water storage. After the first few years the infiltration rate approaches the draught rate, and the depletion approaches the limit 33.68 feet more and more gradually as time goes on.

If a well is put down in the surface ground water horizon, and this is underlain by an impermeable stratum, then the maximum draught from the well will be that for which the depression of the vertex head plus the trumpet head equals the depth of the ground water. Until this depression is attained a greater rate of pumping can be carried on. Formula (3) is therefore adapted to the determination of the maximum yield which can be permanently derived from a given well. For this purpose it takes the form

$$Q = \frac{\pi I}{12}\left(\frac{H_g - h_t}{m}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

in which $H_g$ is the total depth of ground water. The trumpet head $h_t$ is the only factor that varies with the diameter of the well. For a given draught rate, this varies about inversely as the circumference of the bore.

For example, with the other factors the same as before, $H_g = 20$ feet, and $h_t = 5$ feet, $Q = 18{,}300{,}000$ cubic feet per year, or 33,400 gallons per day, a comparatively small yield, because of the limited area of infiltration tributary to the well. This shows the importance of a deep ground water bed as affecting the yield of a well in the surface zone of ground water. The formula has limitations in its applicability to a well penetrating a definite vein of free flowing ground water, in which case $m$ is practically zero.

If a well is put down into a ground-water horizon in a deep permeable stream, the surface of the water table being nearly flat, but there being a gradual flow or drift in one direction, then it is evident that water will flow into the cone of depression from one side, and a nearly equal amount will flow out from the opposite side. The effect will be to distort the form of the area of depression making it eccentric to the well, with the shorter radius on the upstream and longer on the downstream side, but without much influence on the yield of the well. In fact, it is evident that if a series of uniformly spaced wells was put down and pumped at such rates that their cones of influence were tangential circles, then the supply to each well would be derived necessarily from infiltration over the area of influence.

The Miller-Brownlie formula applies directly to this case, whereas the ordinary formulas are inconsistent in that they assume in such a case that the well is supplied from the area between the tangential cones of influence, in spite of the fact that at the points where the circles of influence are tangential there is no infiltration whatever.

*Depletion of artesian basins.* The matter here considered is entirely distinct from the reduction in flow of artesian wells by mutual interference of wells spaced too closely. An artesian basin may have a capacity so great that the pressure over the basin as a whole is not materially affected by a limited number of wells, but if two or more wells are spaced too closely, cones of pressure reduction adjacent to the wells will overlap, and the supply available from each well will be reduced.

On the other hand, a large number of wells distributed over a basin, and so spaced as to avoid mutual interference, may gradually deplete the artesian supply to such an extent that some of the wells, usually those at higher elevations or nearest the intake beds, will cease to flow. This condition has developed notably in some of the great artesian basins of Australia, and in Southern California.

It is convenient here to consider artesian basins of four kinds:

(1) Inclined basins with ocean outlets.

(2) Inclined or saucer-shaped basins with outlets through artesian springs only.

(3) Inclined or saucer-shaped basins without natural outlets.

(4) Buried or imbedded artesian basins.

The various artesian horizons adjoining the Gulf of Mexico in Southern Texas afford illustrations of the first type. Here, before wells were bored, a balance was maintained between the inflow to the basin through the intake beds, and the outflow where the beds are exposed beneath the sea level, by slight fluctuations in the volume of storage in each artesian horizon.

The conditions are illustrated by figure 2. The pipe $AB$ corresponds to the water-bearing beds. As long as the outlet at $B$ remains unchanged in size, the water level at $D$ will fluctuate only to such an extent as is required to satisfy the storage equation

$$\text{Inflow} = \text{Outflow} \begin{pmatrix} + \text{ Gain or} \\ - \text{ Loss of Storage.} \end{pmatrix}$$

If in addition to an outlet at $B$, wells are put down along the line $DB$, affording additional outlets, then a less head will be required to dis-

charge a quantity of water equal to the inflow at *A* and the ground-water horizon adjacent to the intake will fall to some level lower than *D*. As the flow of wells is increased between *A* and *D*, the pipe will not be filled with water and a well put down in this region will have no pressure in excess of that of the atmosphere. As the number of wells between *D* and *A* is increased, the static water level in the ground-water horizon on the intake side of the basin will further recede, the artesian pressure over the entire basin will be decreased, and if the number of outlets is sufficient, the artesian horizon will fall to some level *C*, and wells located between *D* and *C* will cease to flow, while these between *C* and *B* will continue to flow but under greatly reduced heads.

It is evident from these considerations that excessive draught from an artesian basin in general has the effect of reducing the artesian area on the side of the basin located nearest the intake beds. Precisely this result has followed excessive pumping from the artesian basin of Los Angeles County, Cal., illustrated by figure 3, where the intake beds lie to the northeast. In such cases the depletion of the artesian horizon and the reduction in the artesian area may be very gradual, if the basin is large, owing to the fact that during the period of depletion to a given level the available supply equals the simultaneous inflow plus the water previously stored in the water-bearing beds of the depleted area.

Reverting to figure 2, the available supply during depletion from level *D* to level *C* equals the inflow plus the initial storage in the prism *DC*, when saturated, minus the limited volume of water remaining in this prism after depletion of the water horizon to the level *C*.

The lower peninsula of Michigan apparently affords examples of large saucer-shaped artesian basins, in which the natural outflow was through artesian springs in the river valleys. The conditions governing the depletion of artesian areas without ocean outlets are the same as for inclined basins with ocean outlets, but the effect may be somewhat more complicated.

Sealed artesian basins of type 3, for which there are no natural outlets, the water remaining stagnant until the wells are put down, are probably rare in occurrence and usually of limited extent. Such a basin having no natural outlets may, if tapped by a bore, yield at first a large supply of water, but if, as is likely to be the case, the storage in the water-bearing beds and the intake area are limited, the available supply and the artesian head may decrease rapidly.

It is possible that such an artesian bed may have been saturated with salt or connate water, and the effect of an artesian bore in such a basin may be the production of a well from which the salt or other

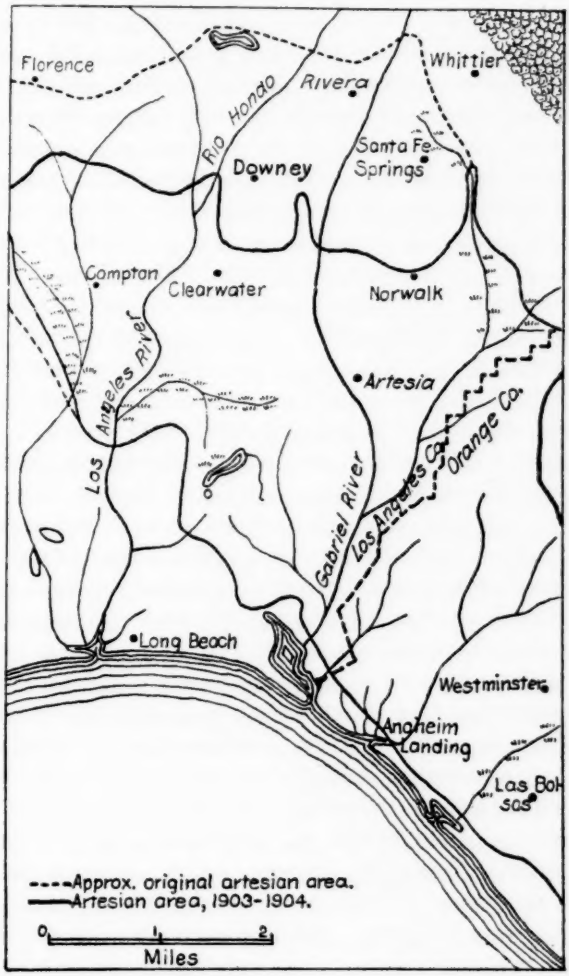


FIG. 3. DOWNEY ARTESIAN AREA, CALIFORNIA

mineral constituents of the water will gradually decrease as the original stagnant water is replaced by fresh water taken in through the intake beds.

Obviously, an artesian horizon without natural outlets, but with an intake bed, would fill up to the surface level at the intake beds. The intake beds in such a case would probably be permanently or intermittently moist or swampy. The draught of water from the basin through artesian bores would have a tendency to drain the swampy intake area. There is some evidence that a few instances of this kind may have occurred in the case of shallow artesian horizons underlying the deposits of glacial drift in Southern Michigan.

It is of course possible that beds of porous material laid down in the bottoms of shallow fresh-water lakes, and around their margins, may be covered with impervious materials while saturated, in such a manner as to be completely sealed, leaving no outlet nor permeable intake area. An inclined bed of porous material bearing connate water, but without an intake area, would, if tapped by a bore under suitable conditions, afford a yield of artesian water which might be fresh but would probably be highly mineralized. Theoretically, at least, air must enter the bed before water can flow out, unless the bore reaches the ground surface at a level more than 34 feet below the uppermost water surface level in the water-bearing bed. However, more or less air and other gases are always contained in the pores of even the most impervious rocks, and air will flow through openings which water will not enter, so that a bed of connate water such as described, and which represents virtually a fossil lake, may yield at first a liberal supply of artesian water. Having no intake supply the flow from the well will rapidly decrease and soon cease, and the supply cannot be replenished.

In the case of artesian basins of the first three types mentioned, having intake beds, the original conditions can be restored after depletion by a cessation of draught from the wells sufficient to restore natural equilibrium by filling up the water-bearing beds to their initial level. It is important to note that repletion occurs only when the inflow exceeds the draught. A reduction of the draught rate to the equivalent of the inflow rate after the artesian horizon is depleted affords no remedy. An equal quantity of water can be drawn from the same basin continuously under greater heads, over a larger area, and in larger volumes per individual well, if the original conditions of equilibrium are restored.

For example, the artesian basins of Los Angeles County, Cal., may be capable of yielding permanently 200 cubic feet per second or more. Under present conditions, as a result of excessive draught,

this volume of water can only be obtained at great cost by pumping from excessive depths. By permitting a restoration of the original storage in the artesian horizon, the majority of the wells would again flow at the surface, pumping could be dispensed with, and a great economic saving would result.

In connection with the use of artesian wells for public water supply, a similar condition sometimes pertains. A proper mode of procedure in such cases is to obtain an auxiliary source of supply adequate for present uses, allowing the artesian supply to rest for a period of years until restoration of the artesian horizon is accomplished. From then on, the artesian supply may be used as an auxiliary to meet increased water requirements, and so long as the draught does not exceed the available inflow to the intake area, the cost of procuring the water will be a minimum.

It is particularly to be noted that in general an artesian supply differs from a ground-water supply derived from shallow wells in the surficial ground-water zone, in that the intake area of artesian supply is fixed by natural conditions and does not vary with the draught from the ground-water horizon, whereas in the case of shallow wells in the surface zone, the infiltration area generally increases as the depth of depletion of the ground-water horizon increases by pumping. As a result, artesian supplies are likely to be lowered by excessive pumping more rapidly than surface ground-water supplies.

The continent of Australia consists of an arid interior basin, surrounded by mountain ridges. There are several great artesian horizons within this basin, the largest, at least, of which is of the interior type with artesian spring outlets. The extent to which agriculture can be carried on in Australia is determined by the water supply. A given number of sheep can be raised per miner's inch of available water. The yield of artesian bores in the Australian basins has gradually decreased since the utilization of these waters began.

The decrease in the yield of artesian bores may be due to various causes other than excessive draught.

(1) These causes include caving in of the bottom of the well, thus plugging the outflow pipe.

(2) Partial closing of the pores in the water-bearing bed, closely adjacent to the well, by silt.

(3) Leakage from the well into overlying non-artesian strata as a result of corrosion of the well casing.

While these conditions are all experienced in Australia, yet accurate measurements of the yield of hundreds of bores indicate that the average yield per well is decreasing at a rate of nearly 8 per cent per annum.

In the Los Angeles County artesian basin (figure 3) the area in which wells flow has decreased from an original extent of 150 square miles, to 105 square miles in 1904. In the Downey district there were more than 3000 wells in 1904, and in the adjacent Las Balsas

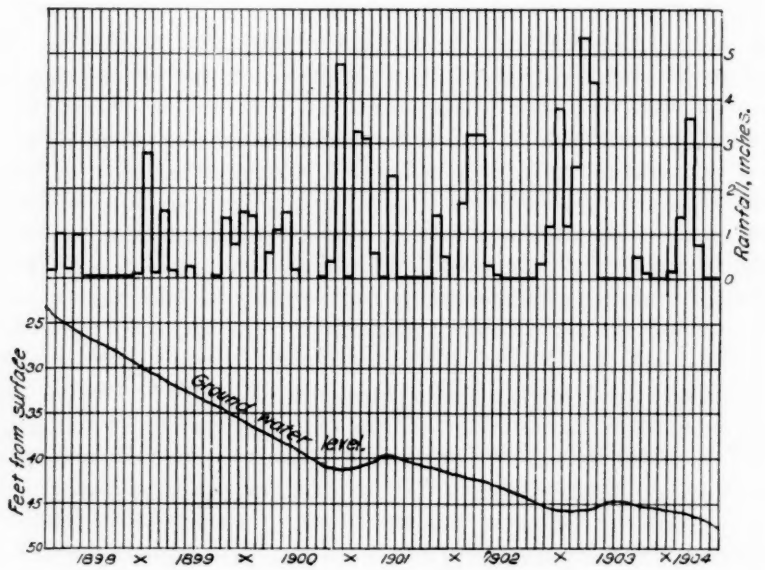


FIG. 4. VARIATION IN GROUND-WATER LEVEL NEAR ANAHEIM, CALIFORNIA (AFTER MENDENHALL)

District 300 wells. At that time a supply of 265 cubic feet per second was derived from pumped wells, and an estimated supply of 500 cubic feet per second was derived from flowing wells, making a total draught from the artesian horizon of some 750 cubic feet per second.

Figure 4 shows the annual rainfall and ground-water level in the artesian horizon near Anaheim, Cal. In general the ground-water level has been lower at the end than at the beginning of each year, in spite of excessive rainfall in some seasons.

*General depletion of ground water.* Many persons believe that there has been a progressive dessication of streams, springs, and ground-water supplies since settlement in this country began. This is commonly attributed to the clearing of woodlands, which are popularly supposed to possess remarkable virtues as conservators of natural water resources. The determination of the effect of a cover of vegetation over the soil on ground water underneath is one of the most complicated and profound problems of hydrology.

It is not the intention here to go farther than to point out the uncertain foundation on which popular opinion is often based in such matters. During the summer season broad-leaved trees intercept about 25 per cent of the rainfall (Ref. 8). Natural herbaceous vegetation and cultivated crops intercept a smaller but still a considerable proportion of the rain falling on them. Clearing away the forests should, and undoubtedly would, increase the rain reaching the ground, and consequently the supply available for infiltration and repletion of ground water. There are other modifying factors.

Evaporation from the soil under forests or other vegetal cover is materially less than from bare ground. The opportunities for infiltration may be profoundly changed as a result of clearing and cultivation. In general the freedom of infiltration to the soil in a tilled field is greater than for any natural soil surface within woodlands or without, excepting bare shifting dune sands. Drainage of swamps and subsoil waters which frequently accompanies clearing and tillage may, however, greatly decrease the amount of deep infiltration. As regards effect on springs and ground water, subsequent treatment of the land, whether by drainage and tillage, or otherwise, must be taken into account, as well as the effect of the removal of forests.

At the University of Wisconsin Prof. F. H. King found that ground water stood generally at a greater depth below surface underneath crops than under bare areas in the same field. This would be expected because of the relatively large amount of water derived from infiltration, which is returned to the air by vegetation through the process of transpiration (Ref. 4).

From an elaborate study of ground-water levels at wells in forests as compared with those in adjacent clearings, Ototzkij found in general a marked depression of ground-water levels underneath woods as compared with those in the same soils and geologic formations in adjacent clearings (Ref. 5).

On the other hand, the late Dr. W. J. Magee, of the U. S. Bureau of Soils, compiled statistics for 28,906 wells, distributed over the United States, from which he reached the remarkable conclusion that since settlement began there has been an average lowering of the surface ground-water table of the United States, outside of the irrigation region, of no less than 9 feet. After examining in detail the published data on which Magee's conclusion is based, the author feels unable to accept this result. The data were obtained by circular letters. Very naturally those holding the affirmative view would be most likely to reply. It is the remarkable instances of such phenomena which attract attention, and there is also a tendency in compiling such statistics towards "counting the hits, and forgetting the misses." From the published data, it appears that many of the affirmative answers are, after all, mere opinions, not based on any measurements whatever, and that as to many others, decrease of the depth of water in wells was probably due to silting up of wells, decreasing their inflow capacity, or their actual depth, or both.

When a well is first put down, the ground-water level is usually measured. If it is ever measured again, it is likely to be in time of extreme drouth, when there is fear of shortage. Thus the two measurements compared by the casual observer are most likely to be, one under original natural conditions, and a later one under conditions of extreme drouth.

The author does not feel that this problem is capable of a general answer for the whole country. What has happened in a particular locality may, however, probably be determined in most cases by a careful hydrologic study of the local conditions. It does not seem that there is evidence to warrant the commonly expressed fear that ground-water supplies will gradually disappear as time goes on.

In conclusion, experience shows extensive depletion of ground-water supplies in some instances over large areas. The analysis of well problems by Miller-Brownlie's method leads to results concordant with experience in such cases. This method depends on the assumption that the ground-water slope in the cone of influence approaches a minimum limit as depletion progresses, and that the supply to the well is derived from storage within this ultimate region of influence plus infiltration over the depleted area.

This differs from ordinary well-yield formulas in which, while it is assumed that there is a maximum limit of the cone of influence, the assumed limit is usually smaller than that used in the Miller-Brown-

lie method. In ordinary formulas, infiltration over the cone of influence is usually neglected, and it is assumed that the supply is derived from ground-water flow from outside the cone of influence. Both methods of analysis are based on convenient working hypotheses. Neither one completely replaces the other, but each has its special applications.

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# DISCUSSIONS

## THE SCORING OF WATER SUPPLIES

The writer has read with interest Mr. Wolman's several articles on "Index Numbers and Scoring of Water Supplies." The systematic collection of these data is certainly commendable. As to the best method of interpretation there may properly be some question. Certainly we should avail ourselves of the aid afforded by mathematics and statistics, and yet every effort should be made to maintain simplicity of expression, and this is indeed difficult. The averaging of results has the further defect of tending to obscure abnormalities, and one abnormal result may be more important than the average for a long period.

The index system proposed is based upon the B. coli test, and it is to the interpretation of this test that these remarks will be confined. Some years ago the writer became interested in trying to reason out what a percentage of positive tests really meant. The ensuing studies led from rather simple premises through rather involved mathematics to the following equation:

$$x = \log_e Q$$

where $x$ represents the number of B. coli per sample, and $Q$ the percentage of negative results in the series of tests made. The two fundamental assumptions made are that the number of B. coli in the water tested does not vary while the tests are being made, and that the number of tests made is sufficient to obtain representative results. This equation is the key for translating percentage results into actual numerical results which can be understood and dealt with in customary ways.

The mathematical deduction has been published elsewhere, but for non-mathematical readers it may be said that the only premise is the well-known "law of averages" as illustrated by the fact that a coin when tossed a large number of times will give "heads" as often as "tails." From this scientific fact, which most of us learned in early school-days, even though it was not in the curriculum, to the

above mentioned formula, the path is one of legitimate mathematics, which the non-mathematician must take on faith, with the further assurance that all mathematical processes are nothing more than short-cut methods of counting. After publishing, the writer learned that M. H. McCrady had published a method of interpreting B. coli results about a year before. A study of the McCrady formula showed that by eliminating certain assumptions as to size of sample, etc., it could be reduced to the above form. A month after the writer's article, a paper on the same subject was published by Greenwood and Yule in England, and again, their formulas can be reduced to the expression above. We therefore have three independent investigators leading to the same result. This covers all the recent work on interpretation of B. coli results, except that of W. F. Wells, which will be mentioned later. The Coli Index of Professor Phelps is admittedly only an approximation.

These matters are mentioned because the existence of several formulas has led to confusion of thought. Actually these are only variants of the formula given herein, into which the several authors have injected special constants, or which they have not reduced to final form. This formula must be the first step in any strictly logical interpretation of percentage results. Since, in the derivation it was assumed that the bacteria do not vary during the series of tests, and such a condition is seldom realized in fact, the direct application of this and all other proposed formulas to practical tests as usually conducted is very limited, and their indiscriminate use in unskilled hands will lead to erroneous results.

A careful study of the basic formula leads to results of utmost importance in carrying out and interpreting B. coli tests.

1. In order to avoid the indeterminate condition represented by 100 per cent. positive, each sample must contain not over 2 or 3 colon bacilli. This means that the results of fermentation tests are on a much lower scale of accuracy than the ordinary plate count; for example, the percentage positive in a water of constant composition determined by means of 200 fermentation tests will give the B. coli content about as accurately as a single plate will give the ordinary bacterial count.

2. Since fermentation tests are usually made in dilutions increasing as 1 to 10 to 100, it follows that only the percentage positive obtained from one of the three sets will lie in that portion of the curve representing the fundamental equation which gives determinate results.

Of the other two sets, one will usually be very close to 100 per cent positive and therefore practically indeterminate, and the other so close to zero as to be also practically indeterminate. Therefore, it is better to choose the most accurate of the three results and discard the other two. Any formula which obtains the average of all three dilutions, very greatly lowers the accuracy of the final result.

3. Since the fundamental formula assumes no variation in the concentration of coli while the tests are being made, it is necessary to inoculate a sufficient number of fermentation tubes at one time to obtain a percentage positive with some accuracy, in other words the "battery" method should be used. Any formula which claims to be able to derive rational results from the percentage of positive tests in a series extending over a considerable period of time, must make use of some occult science. This statement might be modified in the case of filtered, sterilized water, where in last analysis more reliance is placed on the purification processes than on the results of the tests.

As in all cases involving sampling, B. coli results are subject to error, due to the fact that insufficient tests are made to obtain representative results. Once the relationship between percentages and actual numbers has been established, the usual law of errors can be applied to these numbers, and the relative accuracy of results can be established. This the writer has treated so fully elsewhere that it is merely referred to here to point out another use of the fundamental equation.

Even though, as just said, we cannot draw definite conclusions from a series of tests extending over a period of time, still, it is possible to gain an insight into the workings of such a series of tests by assuming that the coli content varies in some definite way, and then computing the corresponding equation of translation. Several such results are shown graphically in figure 1. Curve I represents the fundamental equation with no variation in coli content, the others are plainly labeled, and also shown graphically by small sketches. Please note that it is not necessary for the coli content to pass consecutively through the values in the sketch curves, but may vary in a haphazard manner, provided that the values assume the proper curves when arranged in order of magnitude. It will be noticed that when the coli content follows the normal law, the curve approaches that for the fundamental equation, and since Mr. Wolman's research has shown that the B. coli in filtered water taken over a long period

approximate the normal law of variation, although somewhat asymmetrically, the legitimacy of using B. coli results for index purposes is fortified.

Just as the equation of translation is different for each assumed mode of variation of colon bacilli, so likewise does the probable error of sampling differ, constantly increasing as the variation becomes more pronounced, that is from Curve I to Curve VI. For example, suppose that in a series of 10 cc. tests, 50 per cent are positive. Then on the assumption that there was no variation during the tests, the B. coli would be 0.70 per sample and around this point a circle has

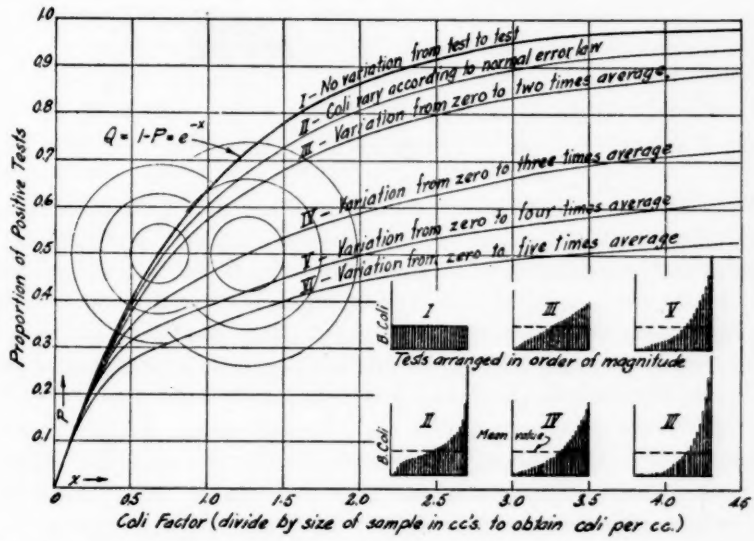
Proportion of Positive Tests
Coli Factor (divide by size of sample in cc's. to obtain coli per cc.)
Q = 1-P = e^-x
I - No variation from test to test
II - Coli vary according to normal error law
III - Variation from zero to two times average
IV - Variation from zero to three times average
V - Variation from zero to four times average
VI - Variation from zero to five times average
B. Coli
Tests arranged in order of magnitude
Mean ratio

FIG. 1. CURVES OF EQUATION OF TRANSLATION FOR DIFFERENT MODES OF VARIATION OF B. COLI

been drawn representing the probable error. Similarly assuming a variation as in Curve IV, the result would be 1.3 per sample, and the probable error as shown. Since an error of three times the probable error is possible, the tests may indicate anything from 0.4 to 5.0 coli per sample, or from 4 to 50 per 100 cc. in this case.

It is not clear from Mr. Wells' several papers how he arrived at the use of the geometric mean as a B. coli index, but apparently it was in the nature of a "discovery" resulting from the study of numerous actual results, rather than by deductive methods. If in an

extended series of tests considered in sets, the proportion of negative tests in the several sets are represented by $Q_1$, $Q_2$, $Q_3$, etc., and it is assumed that the corresponding numbers of B. coli, $x_1$, $x_2$, $x_3$, etc., are obtained from the relation, $x_n = \log_e Q_n$, then the average $x$ for the series

$$x = \frac{x_1 + x_2 + x_3 + \ldots . x_n}{n}$$

but from the fundamental equation

$$x = \frac{\log Q_1 + \log Q_2 + \log Q_3 \ldots .}{n}$$

or

$$\log Q = \frac{\log (Q_1 \cdot Q_2 \cdot Q_3 \cdot \ldots . Q_n)}{n}$$

Therefore

$$Q = (Q_1 \cdot Q_2 \cdot Q_3 \cdot \ldots . Q_n)^{\frac{1}{n}}$$

That is to say, the average proportion of negative tests is the geometric mean of the proportions for the several sets. The same rule holds for the positive tests. There is nothing in this to take account of the time element, that is, the variation of B. coli content from test to test, which is a matter of chance, and cannot be accounted for in any formula, except that in the long run it approaches the normal distribution, as already pointed out.

Mr. Wells, however, applies the geometric mean directly to the number of B. coli, after it has been obtained by the method of Phelps or otherwise. The legitimacy of this seems to the writer doubtful. The geometric mean carries with it the idea of growth, such as the average population of a growing city over a period of time, or the average of a number of platings from a growing bacterial culture, etc. This is the way that the geometric mean and the time element are associated. But in the bacteriology of water, the bacteria found on one day are not the direct descendents of those found on previous days, the variation from day to day being due not to a regular growth, but to changing currents, run-off conditions, etc., and in the case of filter effluents to other equally accidental causes. Therefore it would seem more proper to use the arithmetic mean. Mr. Wells states that bacteriological results follow a logarithmic probability curve, which agrees with the writer's conclusions, first, because of the basic logarithmic equation given at the beginning of this dis-

cussion, and, second, in the case of tests extending over long periods the results are brought about by innumerable causative factors, each acting independently, and under such conditions a normal probability curve results, which is, of course, a logarithmic or exponential curve.

Perhaps, in this discussion, not enough emphasis has been laid upon isolated abnormal results, which the writer has considered elsewhere, including the "coli characteristic." Space forbids more than a mention of the fact that this is a matter of importance.

Mr. Wolman's studies are probably the most thorough made on the B. coli content of filtered water to date. His analytical methods and treatment of the results obtained are sound. It would seem to the writer that it might have been better to use ten tests instead of six per determination, and to use only one dilution. Inspection of the tabular data shows a very asymmetric frequency distribution, probably because the mean is so near the zero axis that there is no chance for a normal distribution without negative values. The reason that the logarithms of the B. coli contents seem to give a more normal distribution is because the small values then come further from zero relative to the large values. This is merely a geometrical condition of no special significance.

It is hoped that this discussion will assist somewhat in straightening out the confusion which seems to exist regarding the methods of interpreting B. coli results.

MILTON F. STEIN.[1]

## BACTERIAL UNIFORMITY INDEX

The close fit of the curves in Mr. Wolman's article on Index Numbers[2] is exceedingly valuable confirmation of the principle underlying the "Geometrical Mean" as a B. coli index. While these are probably the first published results showing the daily distribution of B. coli, the phenomenon was discovered in 1916 from an analysis of such data obtained in connection with oyster studies.

Professor Phelps has thrown light on the problem by distinguishing between the distribution of B. coli in space and its distribution in time. The former alone was discussed by McCrady (*Jour. Infect. Dis.*, 1915, 17, p. 183) in treating of fermentation tubes made from a single sample. The latter furnished

[1] Sanitary Engineer, 6753 Lafayette Avenue, Chicago, Ill.

[2] JOURNAL, November, 1920, pages 927–930.

the data for suggesting the "Geometrical Mean," which was based on a large number of samples taken at different times from single sources, as, for example, given points on a river. (Basis of the Geometrical Mean as a B. coli Index, *Science*, N. S., vol. xlix, no. 1269, p. 401, April 25, 1919.)

Associate Sanitary Engineer Pincus, analyzing a large number of similar results in connection with the Public Health Service investigation of Jamaica Bay, New Haven Harbor and Narragansett Bay, which, unfortunately, have not yet been published, also confirmed the conclusions.

If the phenomenon is correct, which Mr. Wolman's data also confirm, the "Geometrical Mean" follows as a logical B. coli index, and a very simple means of computing it is possible.

The reason the above simple methods give the "Geometrical Mean" arises from the fact that the ordinary bacteriological dilution scale is in reality a logarithmic scale, and the average dilution is the average logarithm or the logarithm of the "Geometrical Mean." A vast amount of published bacteriological data to be considered at some other time proves that bacteriological results follow a logarithmic probability curve, from which it follows that a median value closely corresponds to the geometrical mean. It is equally probable, therefore, that another sample would be greater or less than the geometrical mean. (The Geometric Mean as a B. Coli Index, *Science*, N. S., vol. xlvii, no. 1212, p. 48, January 11, 1918.)

The rule for obtaining the median value of the results which is, therefore, the geometric mean, called the B. coli index, becomes: Revert dilutions and apply Phelps' method. The direct application of Phelps' method to the results gives the arithmetical mean which, because of the nature of the distribution of the results, is always higher than the geometrical mean. The difference between the arithmetical and geometrical means varies directly with the uniformity of the results, and the ratio of the geometrical mean over the arithmetical mean gives a very valuable uniformity index. The greater the degree of uniformity of the results the more nearly do the arithmetical and geometrical means approach equality, and the more nearly does the ratio approach unity.

Applying this principle to Mr. Wolman's figures we obtain the results given in table 1. A study of the table will show that the uniformity index gives a correct expression of the nature of the variation. In the summary of three years' results which Mr. Wolman proposes as a fair case, the uniformity index is almost exactly 0.50. This states that if the geometric mean of the results is required to

pass the U. S. Treasury standard of 2 B. coli per 100 cc. of water, the arithmetic mean should not be more than double that figure or 4 B. coli per 100 cc. of water. This will be observed to be very approximately the case in the three-year averages given in the table.

The trail which Mr. Wolman is following, if persisted in, will give the reason for proposing "The Bacteriological Dilution Scale and

TABLE 1

*Uniformity index of B. coli variation in samples of tap water during three years. Geometrical Mean ÷ Arithmetical Mean = Uniformity Index*

| B. COLI PER 100 CC. | | 1917 | 1918 | 1919 | SUMMARY 1917, 1918, 1919 |
|---|---|---|---|---|---|
| Between | Assumed average | | | | |
| From variation of *B. coli* content of all tap water samples | | | | | |
| 0–2 | 1 | 132 | 146 | 178 | 441 |
| 3–5 | 4 | 63 | 32 | 5 | 94 |
| 6–10 | 8 | 39 | 13 | | 47 |
| 11–20 | 15 | 19 | 5 | | 20 |
| 21–30 | 25 | 7 | 0 | | 5 |
| 31–40 | 35 | 1 | 2 | | 2 |
| 41–50 | 45 | 1 | 0 | | 0 |
| 51–100 | 75 | 3 | 2 | | 3 |
| 100+ | 150 | 2 | 2 | | 2 |
| Total samples | | 267 | 202 | 183 | 614 |
| Geometrical mean | | 2.78 | 1.65 | 1.04 | 1.71 |
| Arithmetical mean | | 6.97 | 4.82 | 1.08 | 3.59 |
| Ratio = Uniformity index | | 0.40 | 0.34 | 0.96 | 0.48 |
| After eliminating abnormal results below line | | | | | |
| Total samples | | 262 | 196 | 183 | 609 |
| Geometrical mean | | 2.59 | 1.55 | | 1.65 |
| Arithmetical mean | | 5.10 | 2.56 | | 2.78 |
| Ratio = Uniformity Index | | 0.51 | 0.67 | | 0.59 |

the Dilution as a Bacteriological Unit" (*Jour. Am. Pub. Health Assn.*, September, 1919, pp. 664–667). With such a direct scale the average deviation of the dilutions positive would represent the uniformity index. Since, however, the mathematical interpretation does not correct for lack of precision of experimentation, a "Standard System of Bacteriological Dilutions" (*Jour. Am. Pub. Health Assn.*, December, 1919, pp. 956–959) has been proposed to make laboratory meth-

ods consistent with the resulting system of computation. "An Improved Fermentation Tube Battery" (*Jour. Am. Pub. Health Assn.*, December, 1918, pp. 904–905) will assist to simplify the bacteriological technique.

*Conclusion.*—"If dilutions are made in conformity with a standard dilution scale, and results computed in terms of dilutions, and interpreted directly according to their location on the scale, many of the present difficulties of bacteriological standards will disappear."

WILLIAM FIRTH WELLS.[3]

## METERS FOR MULTIPLE-FAMILY HOUSES[4]

A private company differs somewhat from a municipal plant in that when new buildings go up in a municipal territory, building plans are generally submitted when an application is made to the water department for service and the department is immediately in possession of facts as to the size of the building, number of families and all information necessary to determine on the size of service to be installed. Such is not the case with a private company. In many cases the advice of our company is sought and almost invariably followed, but in more cases the application comes from the owner and the size is determined by either the plumber or architect.

It is difficult to lay down a hard and fast rule for the size of meters that will be workable in all cases, because there are so many things to be taken into account. The pressure condition is one. A pressure of 40 to 50 pounds will deliver through a ½-inch or ⅝-inch meter a certain quantity of water, but if that pressure is doubled the same rule does not apply. On the system of the Hackensack Water Company, where the elevations vary from 20 to over 500 feet, it is obvious that no fixed rule will fit all cases.

A municipal plant can go ahead and install a small service and set a small meter and if the service is not what it should be, or if any complaints are made with regard to its inadequate capacity, it can be changed and no complaints are made to the Public Utility Commission, for the reason that a municipal plant is outside of the jurisdiction of the Public Utility Commission. Whereas, if a private corporation installs a service which is too small to give a proper sup-

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[4] Revision of oral discussion at Montreal convention, June 23, 1921.

ply it is likely to be followed by a complaint being filed with the Commission.

The practice the Hackensack Water Company has been following for the last several years has met with reasonable success. It has found that a ¾-inch service installation from the street main to the meter and a ½-inch meter furnish satisfactory service to a two-family house. The company does not install services smaller than ¾-inch inside diameter and a ½-inch meter will deliver as much water as a ¾-inch service pipe will pass.

For buildings containing five to ten families, 1-inch services and 1-inch meters are installed and with a pressure of 50 to 60 pounds no complaints are received.

A large part of the Hudson County section of the system is developing rapidly into a tenement district, and buildings housing from twenty to one hundred families are numerous. In the larger of these four 1-inch services with a 1-inch meter on each service has been installed. The pressure is about 60 pounds. The buildings are five stories high and not a single complaint has been received because of lack of service.

Too little attention is often paid to the size of risers in large apartment houses. One might install a service and meter of suitable size, but unless the risers are ample enough to take proper care of the upper floors, or unless they are properly cross-connected, there is danger of complaints from the top floors. With connections such as were referred to, and with two cross connections on the fourth or fifth floors of a five-story building, the results have been so satisfactory that this practice is recommended unhesitatingly.

The installation of a suitable service and meter cannot overcome the effects due to improper interior plumbing. The use of black or wrought iron galvanized iron pipe is sure to result, in a few years at best, in decreased capacity on account of interior corrosion and scaling which finally lodges in elbows or elsewhere, partly choking the supply. When the piping approaches this condition we may expect "red water" trouble, especially so if the analysis shows the water to be low in alkalinity.

D. W. FRENCH.[5]

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